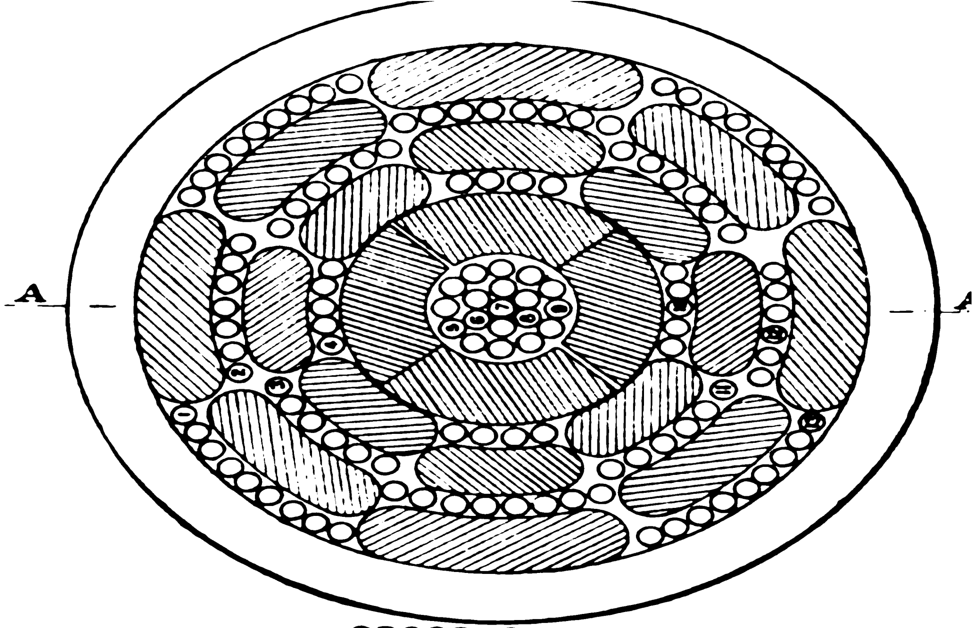

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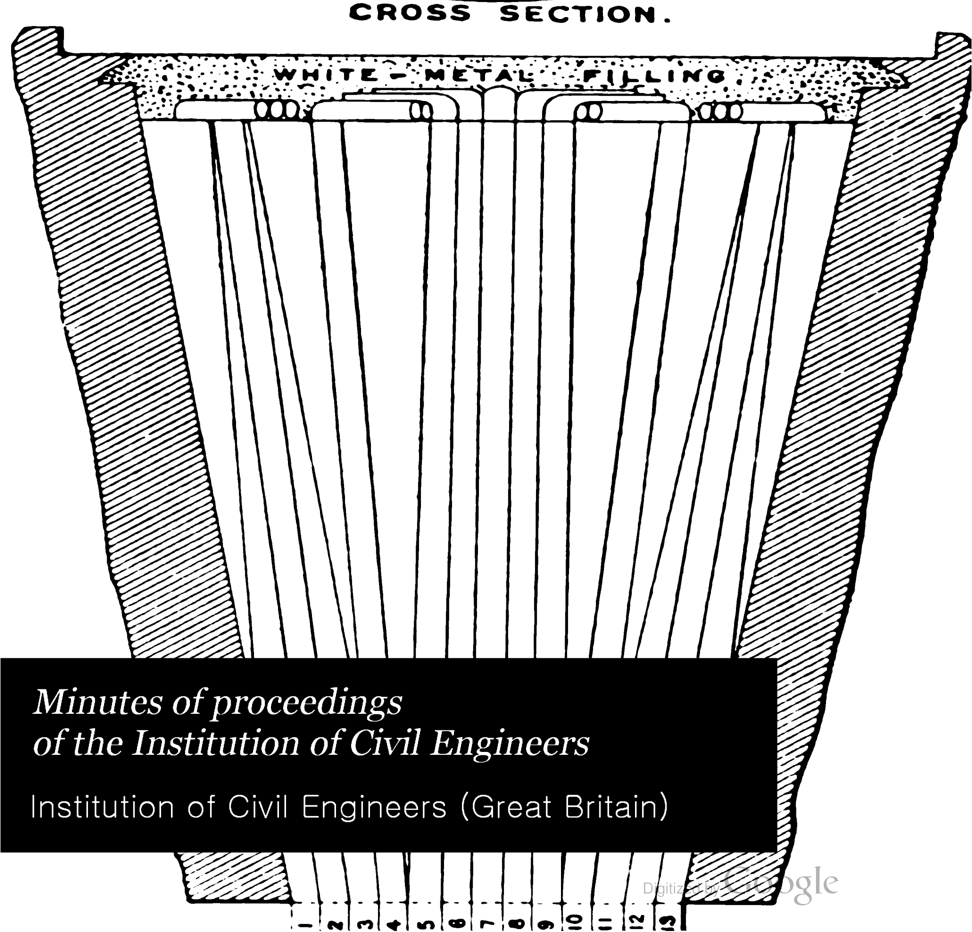

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MINUTES OF PROCEEDINGS

OF

THE INSTITUTION

OF

CIVIL ENGINEERS;

WITH OTHER

SELECTED AND ABSTRACTED PAPERS.

VOL. CLXV.

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CONTENTS.

SECT. I.—MINUTES OF PROCEEDINGS.

20 and 27 *February*, 1906.

6 and 13 *March*, 1906.

20 and 27 *March*, 1906.

SECT. II.—OTHER SELECTED PAPERS.

Sect. III.—ABSTRACTS OF PAPERS IN SCIENTIFIC TRANSACTIONS AND PERIODICALS.

CORRIGENDA.

Vol. clxiv, p. 355, l. 13 from bottom, *for* "transverse" *read* "traverse."
" " p. 401, l. 18, *for* "Baptel" *read* "Boxtel."

THE
INSTITUTION
OF
CIVIL ENGINEERS.

SESSION 1905-1906.—PART III.

SECT. I.—MINUTES OF PROCEEDINGS.

20 February, 1906.

Sir ALEXANDER RICHARDSON BINNIE, President,
in the Chair.

(*Paper No. 3624.*)

"A Plea for Better Country Roads."

By GEORGE ROBERT JEBB, M. Inst. C.E.

THE proper maintenance and improvement of the highways of the country is a subject in which the whole community is interested. Probably the roads are used more now than they ever have been in the past, notwithstanding the development of the great railway system.

As the coaches and stage-wagons were taken off the old turnpike roads, many of these were allowed to get into very bad condition. The by-roads were in a still worse state, the repairs on them being done only when urgently needed, chiefly by farmers; and there was seldom any properly qualified supervision of the work. Very few of these by-roads had been properly constructed; they were usually devoid of a good foundation, and either ill-drained or not drained at all. The traffic on many of the by-roads is now far greater than on the majority of the main roads.

Since the county councils and district councils were established, the main roads in many counties—and notably in some—have been greatly improved; but the improvement has not been uniform: the district roads as a rule—there are, of course, exceptions—are still badly maintained at extravagant cost.

Not only is there more traffic than ever on the roads, but a new kind of traffic has sprung up during the past few years, namely, the motor-car traffic; the pleasure motor-cars are everywhere, splashing through the mud in winter, and raising clouds of dust in summer, causing thereby great discomfort and loss to the public. There are already many heavy motor-cars running on the roads, and the number is increasing rapidly; and heavy mechanically-

propelled vehicles do much more damage to roads and bridges than would be caused by equal weights drawn by horses. These heavy cars are causing considerable anxiety to engineers who have to maintain the public highways and bridges, and it seems therefore to the Author that the present is a specially fitting time for engineers to consider carefully (1) whether they are adopting the most scientific, and therefore the most economical, methods in the maintenance of the existing roads, and (2) what improvements should be made in the roads to render them fit and satisfactory for the new traffic.

It is not the Author's intention to do more than make a few general remarks and suggestions in this short Paper, but he hopes these may lead to a useful discussion, inasmuch as many of the engineers of county and district councils are corporate members of The Institution of Civil Engineers, and are specially qualified by their experience to contribute valuable information on the subject.

The Author believes that if the existing roads were better drained, and if water and mud were not suffered to remain on them, the benefits which the public would derive from having clean dry roads would be much greater than is generally realized. He considers that it is hardly possible to estimate too highly the value of these benefits.

Fairly heavy traffic does but little harm to a hard, dry road, but a soft road is ruined by it. The soft, wet, muddy road seriously increases the cost of haulage; the damage which is caused by such roads to carriages, cycles, boots and clothing is very considerable. In summer the mud becomes dust; and it is now universally acknowledged that the dust nuisance—so greatly increased of late by the motor-car traffic—has become a very serious question. The value of houses standing anywhere near a road has been seriously diminished. If the windows in a house so situated are left open even for a short time the furniture becomes covered with dust: many houses have been rendered practically uninhabitable on this account.

If the roads were regularly kept clean by sweeping—which, in the Author's opinion is, as a general rule, better than scraping—and if certain modifications were made in the design of the underside of motor-cars, the mud and dust nuisances would practically disappear. The roads would soon dry, and the cost of the extra labour would be more than reimbursed by reduced cost in future repairs.[1] When,

[1] The Author ventures to think that, if the streets and footpaths in *towns* were kept clean and dry, many accidents would be prevented, and pedestrians would suffer less from colds and chills. The gain to the public would far outweigh the extra cost of sweeping.

from neglect or other cause, the roads become dusty, it would be better to sweep the dust off from time to time, and to cart it on to adjoining land, rather than to suffer it to remain on the roads and be stirred up by every passing vehicle. Where a road is scraped, the mud is often allowed to remain in ridges on the roadside, and is only removed when it is wanted to "blind" new metalling.

It is still too common a practice, particularly on district roads, to effect repairs with broken boulders picked from the adjacent fields, which are laid on the roads, often in mere patches, covered with soil or mud, and sometimes left unrolled. These rounded stones soon become loose in dry weather, and are then a source of danger to everyone driving along the road.

Special stress is laid on keeping the roads dry, not only on account of its essential importance, but because what is suggested could be put into operation at once on the existing roads, without much reorganization of the present staff of inspectors.

The great question of the improvement of the roads, so as to fit them to bear the wear and tear of present and future traffic, calls urgently for the careful consideration of engineers. It is manifestly impossible to lay down any rules that would be applicable to all roads and all districts. The roads would have to be classified, as they usually are now, into first-, second- and third-class roads; the requirements of each district would have to be considered, and the roads improved by strengthening them on scientific principles, widening and straightening, etc., where necessary, according to special circumstances. The Author suggests that greater care should be taken in the selection of stone for macadam. There is always a temptation to use the stone which can be obtained most readily and cheaply; but it would often be truly economical to pay a high price for a stone that is durable and not friable, the cost of breaking being much the same for one sort as for another. Perhaps limestone, which is largely used in many districts, and almost of necessity in some, produces the most slippery, and for bicycles and motor-vehicles the most dangerous, of any kind of mud, and also the most irritating kind of dust. A little limestone mixed with harder stone would probably be unobjectionable, and even beneficial.

It would be interesting and useful if some of the engineers to county councils would state the results of their experience in maintaining and improving the different kinds of roads under their charge; and if they would also say what particular method of construction they have found to be on the whole most satisfactory,

and, where "tarmac" or other dust-preventing material has been used, whether it has been found to be efficient and durable.

Many people have advocated the making of a number of new main roads. In the Author's opinion the money needed for such work would be more usefully expended in improving the existing roads.

The Author has long held, and has advocated elsewhere, the view that all country highways should be kept in repair by the county councils. Among the advantages of such a change in the present practice would be:—

1. That a properly qualified engineer would superintend the whole of the work.
2. That depots for materials and working-plant could be provided not only at convenient places for particular lengths of a main road, but also for all the district roads within a given distance of it.
3. That there would be more uniformity in the methods of maintenance.
4. That materials could be bought and distributed at less cost.
5. That horses, carts, and all plant, including steam-rollers and sweeping-machines (some of which might be worked by mechanical power), could be more efficiently and economically used.

In illustration of the unsatisfactory way in which district roads are sometimes maintained under the present system, the Author will refer in detail to a case which has come under his observation during the last 12 months, and which is illustrated in *Fig. 1.*

The total length of the road between X and H is less than 1½ mile, and, as indicated in the Figure, no fewer than six public bodies are responsible for its maintenance. The traffic is very heavy, and the road was not originally constructed to bear such traffic; there is practically no foundation, and the drainage is imperfect.

About 6 months ago a length of about 300 yards of the road between X and C was remetalled with good material. The work occupied several weeks, and the steam-roller had to make many long journeys to and from it. If more men had been engaged several of these journeys would have been unnecessary.

A little later on a length of about 200 yards of the road between E and F was covered with broken boulders, blinded with a thick layer of mud, soil and turf: this was eventually rolled, but the surface of the road was soon in an exceedingly muddy condition, and has continued to be so ever since; it has already become

uneven again. The metalling stopped short of the boundary at F by about 10 yards.

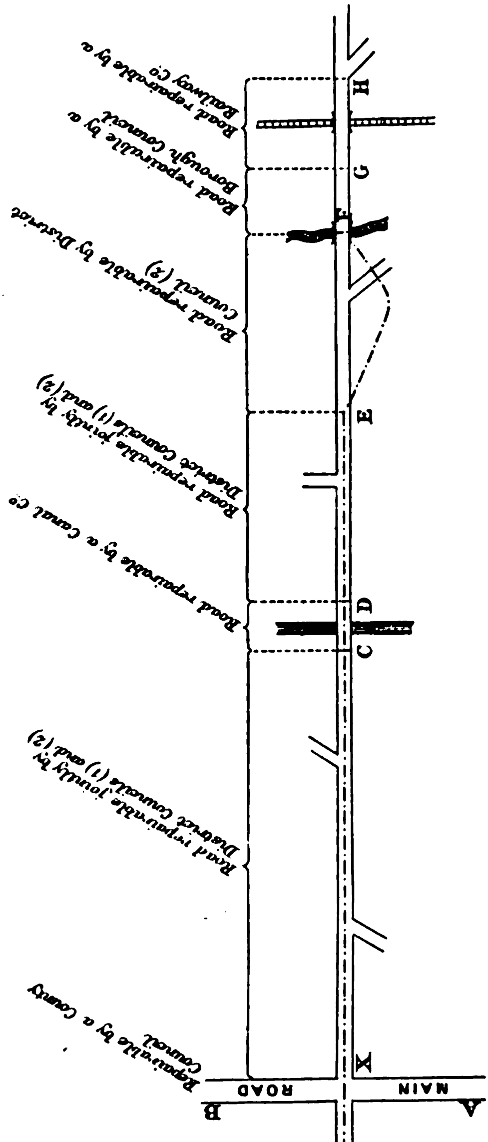


Fig. 1.

Still a little later, a borough council metalled from the boundary at F to the foot of the railway-approach at G (about 150 yards).

This was done fairly well and was rolled by a steam-roller belonging to the borough council. At the point F, on the middle of a bridge over a stream, there is an awkward drop in the surface of the road, caused by the borough council having raised their portion of the road some inches above the level of the road repairable by district council No. 2.

The length of road between D and E has a steep gradient: when heavy loads are being brought down the wagon-wheels are either locked by a chain, or are allowed to slide on a slipper, and the thin coating of macadam has been ploughed through in many places. Two deep and dangerous holes were made, which a few days ago were filled with coarse gravel: in a very short time they will be ploughed out again.

It is clear that there has been great waste of money in carrying out the repairs which have just been described, and some of the work has been very badly done. It would have been done much more satisfactorily, and at much less cost, if it had been carried out by one road-authority, under the supervision of a properly qualified engineer. The aggregate loss entailed by similar methods in the whole country must be enormous.

The Author is of course aware that before the change which he advocates (namely, making the county councils responsible for the maintenance of all country roads) could be carried out, it would be necessary to convince both the public and the Government of the wisdom of such a step, and that an Act of Parliament would be necessary; but if he is right in believing that the change would result in material benefit to the community, and if his opinion is generally supported by the members of this Institution, he has no doubt that any difficulties in the way could be overcome.

If the Author's suggestion that the roads should be better drained and be kept free from mud and dust were adopted throughout the country, it would doubtless be necessary to employ permanently a large number of additional labourers on the roads. As already stated, this work would be, in his opinion, remunerative. If a large scheme for the improvement of the roads, *i.e.*, making them fit for the present and for the probably increased future traffic, were adopted, a further large number of labourers would be wanted for many years to come. The Author cannot conceive that better or more useful work than this could be found for some of the able-bodied men who now swell the ranks of the unemployed. The work would be free from the serious objections which have been wisely raised against many schemes suggested for the benefit of that class, inasmuch as the work is urgently needed in the interest of the general public,

and all classes would benefit by it. One advantage of the scheme would be that the additional men would not be concentrated in one locality, but would be distributed over the whole country.

A Royal Commission on Motor-Cars is now sitting, and the Author hopes that in their report the Commissioners may be able to make some practical recommendations which will lead to the general improvement of the highways of the country: in any case, he believes that a discussion of the subject at this Institution would be of great service to those who will ultimately have to decide what shall be done.

The Paper is accompanied by one drawing, from which the Figure in the text has been prepared.

(*Paper No. 3604.*)

"Country Roads for Modern Traffic."

By John Eaton Blackwall, B.A., Assoc. M. Inst. C.E.

A Royal Commission has recently had under consideration the question of the improvement of the highways of the metropolis to meet the demands of traffic, and the growing public feeling on the subject shows that it will shortly be imperative that similar attention should be paid to the requirements of the present-day and future traffic on the main roads throughout the country.

The gradually increasing use of the traction-engine first forced this matter on the notice of engineers responsible for the proper maintenance of highways; but it was not until the advent of the motor-car, with its noise and clouds of dust annoying every foot-passenger on the roads, that the reform of country roads became a matter of widespread interest.

It may be taken now as a fact, whether the majority of the public like it or not, that haulage by traction-engine is cheaper and more reliable than horse traffic for all heavy work, provided the roads are fit to carry it; the use of the traction-engine is therefore bound to increase, and highway-authorities must make up their minds that the strength of the roads must be proportionately augmented to meet the requirements. A proof of this may be found in the growing difficulty experienced by highway-authorities to prove a claim against traction-engine owners for extraordinary traffic.

The motor-car may be reasonably expected to become a more and more popular means of locomotion as its initial cost and the expense of its maintenance are reduced as the result of longer experience of its manufacture and keener competition, and when such roads are provided that the motorist can start from his home and travel any distance at any speed of which his car is capable, without the inconvenience and delay experienced when travelling by railway.

A good country road to-day provides moderately satisfactory accommodation for pedestrians, cyclists, heavy carts and carriages, provided the traction-engine and the motor-car are absent. No one is so sanguine as to think of attempting to provide a road absolutely satisfactory for all these classes of traffic. The roads were originally made for the four classes first mentioned (with the exception of the cyclist), and they have a prior claim to accommodation on the highways of the future.

The question to be solved is—How are roads to be constructed so that traction-engines will not render them impassable for all other traffic in winter, that motor-cars running at high speed may not endanger the lives of all users of the highways, including their own occupants, by skidding on greasy mud, colliding with other vehicles, persons or animals, or failing to turn sharp corners, and may not envelop in dust during the summer everyone and everything within 20 yards of the road?

A suggestion has been made to provide for motor-cars running at high speeds by laying out main routes through the country for motor-traffic only, on much the same lines as railways, and connecting them with the present roads, but not crossing the latter on the level. Thus a motorist on leaving home might follow the ordinary road at a limited speed until the nearest motor-route was reached; he could then travel, at a speed limited only by the capacity of his car, to that point on the motor route nearest his destination, to which he would then proceed by ordinary road.

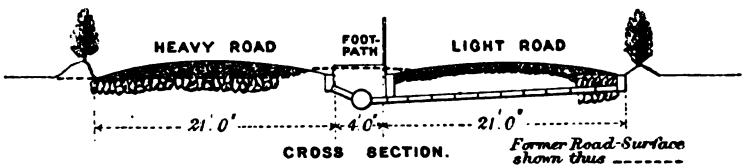
The objections to providing enough motor-routes to accommodate all who might desire to use them are many and obvious: that of initial cost alone suffices to dismiss the idea from practical consideration for the present. The dust nuisance, and the several, at present, abortive attempts to abate it, is in itself enough to convince highway-authorities that a considerable motor-traffic cannot be carried by country roads as they are now constructed.

The most practicable method of making a road which traction-engines will not break up after frost, and the binding material of which will not be drawn out and diffused in clouds of dust by large pneumatic tires, appears to be the use of "tarmac" or some similar combination of the waterproof and unfriable properties of tar with a strong material which will absorb and hold tar without crushing under a heavy or sudden load, or wearing to a slippery surface. The initial cost of some such material will surely have to be borne where busy traffic converges on country roads. In open country, however, where there is a good width between the fences,

or where the adjoining land is not of high value, the Author suggests the following simple and inexpensive form of road.

The road is divided along the centre into two separate tracks, differently constructed, for different classes of traffic. A footpath divides these two tracks, one of which is for light fast traffic, the other for slow heavy traffic (*Fig. 1*). A wire fence separates the footpath from the light road, protecting foot-passengers from being run over by motor-cars in the dark, and freeing motorists from such obstructions as cattle, sheep, dogs, children and, to a certain extent, fowls. By-laws would forbid any person to drive a cart, cattle, sheep or pigs along the light road. Pedestrians would use it at their own risk. Any horse-drawn vehicle moving at a slower rate than a trot would have to be driven along the heavy road. This would apply particularly to hills, at the top and bottom of which turn-outs would be provided, so that a carriage could be trotted

Fig. 1.



along the light road on the level and transferred to the heavy road to proceed at a walking-pace up or down the hills.

The traffic of the light road would consist of motor-cars, bicycles and light carriages. That of the heavy road would consist of carts, wagons, traction-engines, motor-lorries, led horses and all kinds of cattle. The minimum width of the two roads and footpath together is 46 feet between the fences—21 feet for each road and 4 feet for the footpath; but the road should not be restricted to these widths unless circumstances rendered it imperative. The additional expense of buying a strip of adjoining land to widen the road would be repaid in most cases in the saving on road-maintenance in 5 to 10 years, by allowing the heavy traffic to use such extra width.

The surface water would be conducted into a pipe-drain beneath the footpath by pipes near the surface under the light road, and by cross gutters over the heavy road, except where it naturally runs off one side into the adjoining land owing to the general slope of the country, both roads having their crowns at the centre and sloping to each side.

The light road would be formed with evenly broken stone of small

gauge very carefully laid and rolled. The heavy road would be made of large-gauge material of a tough quality, worked in by the traffic where that can do so, *i.e.*, where there is much traction-engine or motor-lorry traffic. Where the traffic is such that the material is loosened by the horses' hoofs, and does not become consolidated in a reasonable time, long lengths would be coated with stone at the period of the year when the surface softens, and would be rolled, binding material being added if the stone had not become consolidated by the middle of February.

The light road would not need to be recoated under average circumstances for between 6 and 12 years, and where possible it should be recoated with tarred material, which will last without renewal for a period at present unknown, possibly for 20 years. Until it was recoated with tarred material it would not be dustless, but having been carefully made with the best material and relieved of all heavy traffic, although the dust-raising influence might be there the dust-making property would be absent.

Time and experience alone can show whether the dust nuisance will continue to such an extent as to make the twin-road system so little better than the present one as to be not worth its initial expense. Experience has proved already that the broad pneumatic tires of heavy motor-cars travelling at a high speed draw out the small particles which form the binding material of a macadamized road; but that a fair proportion of light iron-wheeled traffic tends to consolidate the material and to prevent disintegration: also, that it is the continual grinding action of heavy weights on iron wheels which originates the dust, by chipping and wearing away the stones which constitute the macadam. If the cause of dust is removed, the root of the evil is attacked. The heavy road would not be maintained with a highly finished surface, and in summer it would be dusty in the same degree as any heavily-worked road was 10 years ago; but it was only the motor-car that introduced the dust nuisance, and, with that class of traffic eliminated, the nuisance might be considered to be abolished.

The details of construction of the twin road would vary considerably according to local circumstances, but speaking broadly the following considerations would apply. In converting an existing road into a twin road the metalled portion would have to be widened; the ground newly metalled should be on the side of the heavy road where settling and unevenness could be corrected from time to time with a minimum of inconvenience. The light road when once made up and opened for traffic should be disturbed as little as possible.

Where one side of the road was supported by a retaining-wall the light road should be on the side next to the retaining-wall, so that the greater weight might be on the firmer ground, and that the surface water might be drained over the heavy road and under the light road.

If, before its conversion into a twin road, the original road were allowed to wear in the middle, very little re-forming of its shape would be required to bring the crown of each of the two roads into its proper position; and the neglect, in such case, would result in some saving of cost in maintenance. On the side on which the road was widened, the turf might be used to form a sod edging to the footpath, the middle of which might be made up with the material removed in the process of widening; but the widening should be completed and the heavy road made up before the footpath was formed and the two classes of traffic were separated.

The two roads would, of course, converge and become one at the entrances to villages or any confined spaces where there was not width enough for the twin road, also at places where frequent crossings would reduce the advantage of a twin road. All crossings of roads would have to be notified by sign-posts some distance away in both directions, and the speed of all traffic would have to be reduced to a safe maximum. At gateways into fields on the side of the light road, a space would be left in the wire fence, closed by a rail pivoted on a post near one end and opening upwards, assisted by a counterpoise at the other end. This rail raised into a vertical position would constitute a sign to the traffic on the light road for a reduction of speed. A by-law would make it an offence to leave a gate open. On the rare occasion of a field-gate being used in the dark, the user would be responsible for signalling, by means of a light, his presence on a light road. At a gateway where much heavy traffic crossed the light road, the road would have to be specially constructed to carry it.

In discussing the conversion of an ordinary road into a twin road, the first question asked is, "What would it cost?"

The maintenance of a road on which there is at present enough traffic to make it worth while to convert it into a twin road may cost anything from £60 per mile per annum upwards.

Initial Cost.—Say the width between fences had to be increased from 30 to 46 feet. To move a thorn hedge costs about 1*s*. per yard. If circumstances cause it to cost more than this, the old hedge had better be grubbed up and burned, and a new quickset hedge planted between temporary wooden fences which would have to be kept in

repair until the hedge grew up. Dry stone walling can be moved at 10*d*. per yard. The sod and soft earth between the new fence and the old road would be removed, and a bank would be formed to the new fence, superfluous material being piled up to be afterwards used to make the footpath. The amount to be removed would of course vary very considerably. An average depth of 9 inches would run to $1\frac{1}{3}$ cubic yard per lineal yard, costing 2*d*. per lineal yard. Hard grit-stone, old wall-stone or furnace-slag, of 6-inch to 9-inch gauge, would then be tipped. Two loads per lineal yard would be about the right proportion in the case under consideration. This should be hand-packed on edge, inclined to the vertical, in courses laid normally to the direction of the road.

A 3-inch thickness of 3-inch stone or slag should next be spread evenly, to bring the road to its required form, and rolled dry. If it seemed likely to work loose, it should be blinded with chippings, or well-broken road-scrapings, 2 or 3 years old, should be swept into the interstices. A ton of 3-inch slag should coat $1\frac{1}{2}$ yard of road-length. If the slag or stone cost 5*s*. per ton delivered on the road, and the foundation-material costs 2*s*. per load, this part of the work would come to 9*s*. 4*d*. per lineal yard, including labour and rolling.

The traffic would all be turned on to this newly-made road, and the other half would be taken in hand, for the light road. The surface would probably need to be lowered at each side and possibly some earth would have to be taken out beyond the limit of the old road, to be replaced by foundation-material. The footpath should be made up, the edges being carefully built by hand with sods, and the centre filled in with the superfluous earth, well rammed and finished off at the top with the old road-material hacked up to form the light road. The labour for these operations would probably cost 1*s*. per lineal yard. The light road would receive a 3-inch coating of $2\frac{1}{4}$-inch to $2\frac{1}{2}$-inch road-material, carefully spread and thoroughly rolled, followed by a 2-inch coating of granite of $1\frac{1}{2}$-inch gauge, consolidated to an even surface by rolling with slurry made from carefully selected road-scrapings. The making-up of this road would vary in cost with the distance that the stone had to be brought. Putting it at 9*s*. 6*d*. per lineal yard, it would prove economical to use "tarmac" if it could be done at 14*s*. per lineal yard. The fence would consist of four strands of galvanized wire, one of No. 4 S.W.G. and three of No. 6 S.W.G., carried by T-iron standards 4 feet high above ground, 9 feet apart, and driven 2 feet 6 inches into the ground. This would cost 7*d*. per yard in large quantities.

Assuming that the land is given by the adjoining owner for the public good the initial cost will thus be:

	Per Lineal Yard. £	s.	d.
Moving the fence	0	1	0
Removing sod and earth	0	0	2
Foundation stone, delivered	0	4	0
3-inch ,, ,, ...	0	3	4
Labour in packing, spreading and rolling	0	2	0
,, ,, making the footpath	0	1	0
Making the light road	0	9	6
Fencing	0	0	7
Total . . .	£1	1	7

Cost of Maintenance.—This would vary mainly with the cost of material in the particular locality, and the weight of the heavy traffic. Judging from the scanty experience available of main roads kept in proper repair for heavy traffic only, £32 per mile per annum appears to be a common cost; while for a light carriage-road only, very few examples of which exist, and which needs to be coated only once in every 7 to 10 years, £23 would probably be somewhere about the average annual cost per mile.

The data for these estimates are unreliable, and the figures may be wide of the mark, but the advantages of the system are obvious; and even should the alteration cost £2,500 per mile and the subsequent maintenance twice that estimated for a twin road, the adoption of the system would be economical and would conduce to the benefit of the public.

The Paper is accompanied by one drawing, from which the Figure in the text has been prepared.

Discussion.

The President. The PRESIDENT, in moving that the thanks of the Institution should be accorded to the Authors, observed that at the present time the subject brought forward was more or less prominent in the mind of every engineer, whether in town or in the country.

Colonel Crompton. Colonel R. E. B. CROMPTON remarked that the subject under discussion might almost be said to be the subject of the day. The advent of "motorism"—to use a word coined by Lord Justice Fletcher Moulton—had brought the use of the roads into such prominence that the matter must be dealt with by the chief authority on the subject of road-making and road-maintenance, which he considered was The Institution of Civil Engineers. For a long time past it had been well understood by railway men that the rail formed part of the machine—that the rail and the locomotive could not be dissociated. To those who were studying the problems connected with motorism it was equally evident that the road formed part of the machine, and must therefore be considered from a standpoint entirely different from what had been customary since railways came into general use, and roads became more or less neglected. In fact, since the advent of railways, roads had taken a secondary place, and had been considered hardly worthy of the attention of engineers. He believed that since the days of Macadam and Telford little had been added to the knowledge of road-construction: bridges and other details in connection with roads had been improved, but not until the advent of motorism, with its disadvantages of dust and mud, had it become evident that the matter required to be re-studied. In the old coaching-days the roads were in excellent condition, and now engineers, with all the appliances of modern science, might do something to make English roads worthy of the time. There was no longer any doubt that roads were the secondary arteries of the country. The railway-companies were beginning to appreciate this fact, for far-sighted companies were now feeding their passenger-traffic by means of motor-omnibuses, and he believed that the goods-traffic would be similarly fed by motor-wagons or traction-trains. There were, too,

Colonel Crompton. ominous signs that the railway-companies who wished to have their lines fed with traffic were already to a certain extent at war with the road-authorities in various parts of the country, on account of the defective condition of the highways. Several new inventions relating to road-surfaces had already appeared: these were of two kinds. The first method was the armouring of the surface of the road with a new kind of paving introduced widely in Germany, called "Kleinpflaster." That would apparently furnish one solution of the problem; but it was not yet known whether it was suited for heavy traffic, or for light traffic only. The second kind of modified road-surface included the numerous waterproof roads, of which one example was called "tarmac." These roads did not depend for their consolidation on the use of chips of the same road-metal rolled in with blinding or hogging material well watered, but were made with suitable material, and impregnated under carefully-arranged conditions with some kind of tar, so that when plastic they could be rolled and formed into a close mosaic; they were quite waterproof, and could be repaired from time to time, as asphalt was repaired, by the addition of thin coatings of the same material. He thought the members were much indebted to Mr. Jebb for calling attention to the cruel way in which the roads of the country were now maintained. What Mr. Jebb said with regard to the 1½ mile of road repaired by a number of authorities might be said of many such pieces of road, and had been repeated many times by the witnesses who gave evidence before the Departmental Committee called together in 1905 by Mr. Walter Long. That Committee had reported that the present state of affairs was intolerable, and that it was impossible to improve the roads of the country without further legislation. He was glad to say that the signs of a desire on the part of county councils to improve the roads, and of willingness to carry on experimental work in connection therewith, were very encouraging. The Kent County Council had voted a considerable sum of money towards testing several methods of making waterproof and armoured roads on one of the main thoroughfares near Maidstone, which carried every kind of traffic. The Council had called to their aid a committee of engineers who had paid attention to the subject, of which committee he was glad to be a member; and he believed that from this public-spirited action of the Kent County Council information would be obtained which would be of immense value to the community in general. Mr. Jebb thought that most of the difficulties with regard to roads could not be removed without the use of these new inventions; but Colonel Crompton would go so far as to say that roads were now made in certain parts of England which were

Colonel Crompton.

practically fully satisfactory for all kinds of traffic; all that was necessary was to follow the specification laid down by the first great road-makers, Telford and Macadam, and to use a proper hand-packed foundation, suitably drained, and with metal covering dealt with in the proper manner. There were a few kinds of road-metal used in England which, under certain conditions, did not produce dust or break up under traction-engine traffic, and which in other respects were quite satisfactory: he alluded to the various basalts used in the North and West of England. To show what could be done without using any of the modern inventions when engineers who had thoroughly studied the subject were given reasonable scope, and how the condition of some of the roads of Hampshire and adjoining counties could be completely remedied, he might mention that, a few years ago, a War Office Committee, of which he was a member, was much bothered by the complaints of county councils as to the damage done by War Office traction-engines, which had just returned from South Africa, and which were exceptionally heavy, to certain roads leading to the Borden Camp near Farnham. The War Office took a wise course and obtained the most competent man they could to advise them, and he redesigned the road in question. He succeeded, at a cost not exceeding the £2,500 per mile mentioned in Mr. Blackwall's Paper, in transforming that road into one which was practically indestructible: it had now worn for some years without any signs of deterioration, and with the minimum of dust, under very heavy traffic. The twin road proposed by Mr. Blackwall was enticing, but he regretted to see that in his design Mr. Blackwall had perpetuated a serious error. The excessive camber or curvature of the road was a serious thing to all constructors of modern motor-vehicles. It was the most difficult matter that the designer of the present motor-omnibus, with its double tires, or the road-wagon with the 12-inch tires prescribed by the Local-Government-Board Regulations, had to contend against. It was impossible to design a vehicle to do justice to itself, or to the road, with such a camber as was shown in *Fig. 1* (p. 10), which seemed to be about 16 inches in 21 feet, whereas it ought not to be more than 3 inches. With such small camber it was possible to get perfect drainage, and vehicles could be designed in a reasonable manner; whereas he defied any designer to produce any kind of wheel, having its axle in one piece, which could accommodate itself sufficiently to the surface of the road to prevent it from cutting the road into heavy ruts after a few passages. That excessive camber existed in the London roads to a large extent, and was one of the chief causes of dangerous side-slip. A motor-omnibus, for

Colonel Crompton.

instance, when it got into the gutter could not get out again easily, owing to the camber of the road. The north-to-south road across Hyde Park, which he had lately had to examine closely, had a 20-inch camber in about 40 feet, and he had looked into the matter to see why the principle of road-construction had there been so completely violated. He did not know the cause, but he strongly suspected the following to be the history of the road. Examination showed that the original foundations consisted of brick rubbish, which was satisfactory so long as the road was used by very light vehicles. But the road-authorities had taken to rolling the road with a very heavy roller, with the result that the brick rubbish, which was not hand-packed, and consequently had no arch or key in it, was crushed to dust. A trial hole made in that road had shown that there was a tolerably thick coating of metal, and underneath it a layer of fine brick-powder. It was evident that that was no foundation at all. It did not distribute the loads, and the only means of doing this was by increasing the thickness of the metal itself. Where that metal ought to be 6 inches or 8 inches, it was now about 16 inches thick; the extra metal had been piled on the centre of the road, and had deformed the road to such an extent that the traffic had always to keep on the crown, to which all the wear and tear was confined, the flanks of the road getting no wear at all. A few years ago, at one of the British Association meetings, what appeared to him to be a remarkably good proposal was made by Mr. A. R. Sennett, which he had never heard repeated, namely, that for urban roads the idea of a convex road might now be relinquished, and that it would be a better plan to incline the roads slightly towards the centre, and to put in a continuous central culvert, with a grating, and provide means of washing the roads from the curbs to the centre. In that way many difficulties would be overcome. Instead of the kennel being always filled with liquid mud, which was splashed over pedestrians by passing vehicles, it would become the cleanest part of the road. Washing the road at intervals during the day could be carried out by providing permanent means of flushing in the curb, and the central culvert, with its grated top, would be accessible, and could be laid to any desired incline, whatever the gradient of the road might be. Thus a road might be obtained which could be kept clean all day long, and which would get rid of side-slip. Although mechanical means of haulage might in time entirely supersede horse-haulage, this would not occur for many years to come; and so long as the horse was on the road there must always be dirt which, worked up into a greasy mortar by passing vehicles, would cause side-slip.

Mr. C. H. Darbishire, having been Chairman of the Highways Committee of the Carnarvon County Council for about 9 years, was much interested in the question of road-making and maintenance. Mr. Blackwall seemed to pin his faith to twin roads, but his arguments were supported by an estimate which was somewhat inadequate, and the data on which the estimate was based were admitted by him to be unreliable. Mr. Jebb, on the other hand, would improve the existing roads gradually. It seemed to Mr. Darbishire that in the first suggestion sufficient notice had not been taken of the large expenditure which would be incurred in constructing a road specially for rapid motor-traffic. Many of the existing roads were not laid out as they would be if a new arrangement of intercommunication had to be made. Centres of population had sprung up since the existing roads were constructed; and other centres, which possibly the roads were constructed to provide for, had diminished in importance. Having regard to the speed and power of the modern motor, flatter gradients, although they might involve somewhat longer routes, would in many cases be more convenient. Again, county councils were confronted with the problem of finding funds to carry out the national system of elementary education; and therefore, as long as the existing roads served their purpose—with possibly a little extra expenditure for slight improvements in maintenance or construction—county surveyors would be obliged to cut down their estimates as much as possible. He was strongly of opinion that the plan suggested by Mr. Jebb was the one that would be looked upon with the most favour by public authorities. Then came the question what was the best material for improving the surface. The cheapest covering when sets or wooden blocks or asphalt were inadmissible, was a coating of macadam. Until some means of using macadam was discovered which would eliminate the dust, ordinary macadam with ordinary blinding would continue to be used, having regard to its general efficiency and especially its economy. The dust nuisance was caused by the blinding and the detritus of the macadam itself, which was crushed and pounded by the heavy and slow traffic, being sucked up by the india-rubber tires of fast motor-cars and thrown behind. In a macadam road every stone which formed the layer of macadam, however thick that layer might be, was subject to deterioration by being rubbed against adjacent stones. Therefore for ordinary macadam it was desirable to have the hardest material obtainable, even if a little extra had to be paid on freight. But however hard the stone might be, if the material for blinding was mud, as it unfortunately often was, or sand, or clean chippings, even when Mr. Darbishire.

Mr. Darbishire. rolled together into a solid mass, the stones were never so tight but that they were able to rub against each other. Many experiments with what was called "tar-macadam" had been made, in order to find some substitute for the ordinary blinding material; but the substitutes discovered hitherto were more or less expensive. The use of the word "tar" in connection with macadam conveyed rather a wrong impression as to the material that was generally used. The best results were obtained with pitch mixed with certain oils, which together formed an elastic and durable matrix. It was sometimes said that this matrix, when mixed with limestone or with broken slag, made the most satisfactory covering for a road, because the matrix adhered to the limestone or broken slag better than anything else; but that of course depended on the composition of the matrix. In the previous week, through the courtesy of the City Engineer of Liverpool, who was experimenting in that direction, Mr. Darbishire had had the privilege of examining some of the experiments made with tar-macadam. A matrix was produced which would adhere so firmly—not only to limestone but even to the hardest stone that could be obtained, such as granite—that it was impossible to scrape it off. He saw several lengths laid with that particular matrix, also some with tarmac and various kinds of limestone material, and others with granite or syenite. The harder stone, as in the ordinary macadam road, gave the best results. The same traffic ran over all the experimental sections of the road, and the particular length of granite referred to had been laid for 6 years, during which time it had shown very little sign of wear, while the other lengths had not lasted nearly as well. Before reading the Papers, he had concluded, wrongly as it appeared, that "country" roads in the titles meant other than main roads, and he had therefore issued 1,200 copies of a circular containing a few questions, with a view to learn what really was the condition of the secondary roads throughout the kingdom. He had received a large number of replies, for which he had to thank those gentlemen who had been good enough to answer. It was impossible in the discussion to give the results of the inquiry, but the answers were very interesting and showed, curiously enough, that in many parts of England no serious effect was found from ordinary motor-traffic, but that heavy motor-traffic or traction-engine traffic caused a great deal of damage and much expense. It seemed to him that the best thing for country road-maintenance in the future was macadam laid in a matrix which, while being impervious to water, would give the necessary foothold, and would be fairly inexpensive.

Lord Montagu of Beaulieu.

Lord Montagu of Beaulieu agreed with much that had fallen from Colonel Crompton, especially his reference to concave roads. On his own estate in Hampshire he had tried that plan in a small way, and he had found concave roads answer admirably. The only difficulty he had discovered was that where there was much rain it was necessary to be careful that the middle drain did not become choked. It should be remembered that instead of two drains there was only one in the case of a concave road, and the drainage must be adequate to carry off not only the water but also the smaller detritus. The particular Hampshire road mentioned by Colonel Crompton, to Borden Camp, had been made by the County Surveyor, Mr. W. J. Taylor, M. Inst. C.E., not by Mr. H. H. Humphreys, Assoc. M. Inst. C.E., to whom he thought Colonel Crompton had alluded in his remarks. It was no doubt one of the best roads ever made, but it was so expensive that he did not think the county would ever allow the Council to make a road of that class again: it had already cost about £28,000 for 15 miles, and was not yet finished. Towards that huge expense the War Office, which began by asking for the road to be made adequate for their traffic, and which had led the Council on with hopes that it would contribute handsomely, had ended by paying to date £5,000 out of the £28,000 expended. Consequently, county ratepayers, while admitting that good roads were estimable things, naturally preferred to pay rates on a more moderate scale, and to see experimental work in connection with the road-making carried out elsewhere. There was no doubt whatever that motor-cars would form the bulk of the traffic using the roads in the course of the next 10 years, and after 10 years they would form 90 per cent. of the traffic, and perhaps even more. Already, according to some statistics he had taken for *The Car* last summer, on the Great North Road, the Ripley Road, the Bath Road, and other main roads out of London, a good deal more than one-half of the traffic on an average day was motor-car or mechanically-propelled traffic of some kind. This showed that the roads must now begin to be adapted to the new form of traffic. The only way in which that could be done without much cost at first—putting aside the widening and strengthening of bridges, and more expensive alterations—was by introducing some material into the roads which would render them dustless. He had made numerous experiments in the endeavour to find out the kind of road most free from dust, and he had no hesitation in saying, after trials extending over 2½ years, that the use of tar or tar derivatives was the only remedy in sight at the present moment. Tar could be used in two ways. One was simply using it with ordinary macadam, as on the short length of road

Lord Montagu of Beaulieu. opposite the Savoy Hotel, a piece which had lasted extremely well—far better than the macadam on each side of it, which was laid at the same time. The other way was using what was called "tarmac." The name was misleading, as the material had nothing to do with any form of granite. It was made from iron-slag, impregnated with tar. It was not really a hard material, being somewhat yielding; but if taken out of the furnace hot, and plunged into tar, it absorbed the tar into its interstices, and became practically homogeneous. It was an excellent material. He had recently laid ½ mile of it, and had found that it answered admirably and withstood wear and tear very well. The Highways Committee of the Hampshire County Council, of which he was a member, had put down 750 tons of tarmac in the Aldershot district, where it had answered very well. But it had only been down since September or October, 1905, and therefore it was yet early to say anything as to its durability. At all events, those experiments and others had convinced him that the use of tar in connection with road-material furnished the best solution of the dust question. Alteration of the shape of motor-car bodies, or other measures of that kind, were minor matters, not to be compared with the real solution, namely, making the road of such material that it would be waterproof, mudless and dustless. As to the cost, he found that tarmac cost about 2*s*. 6*d*. per ton more than ordinary granite or basalt; and considering the greater wearing-power of the tar material he thought that extra cost was very moderate. He believed tarmac would be the most economical material to use when making a new road, or re-coating an old one. With regard to heavy traffic near towns, he could not help thinking that, where there were traction-engines of 14 or 15 tons and upwards it would be necessary in the end to go back to the plateways of 70 years ago. Probably that would be the only solution for very heavy traffic, because a steel plate properly laid would last a practically unlimited time, and it seemed to him to be the only thing that would not be crushed by the heavy weights which future traffic would no doubt put upon the roads.

Mr. Fowler. Mr. A. M. Fowler observed that, having had charge of roads in many large districts, he had been struck, on reading Mr. Blackwall's Paper, by the point Colonel Crompton had raised, namely, the curvature of the road. In constructing macadam roads in suburban districts it was not only the camber of the road that had to be borne in mind, but also the distribution of the traffic over the road. If a road was seriously curved, drivers naturally kept to the middle, and the sides of the road were not utilized. When

the middle became much worn, the drainage became defective, and pools of water added to the drawbacks of the road. He had always found it a good thing in towns and suburban districts to water macadam roads at night, so that the water would sink into the interstices of the metal; then the traffic did not work up the material between the metal, and so damage the road. When a road was watered in the daytime, wheels licked up the surface in large patches, and sometimes actually the stones themselves. He did not quite see how two roads could be made in a country district as proposed in Mr. Blackwall's Paper. It would be rather difficult to keep them in perfect order; and the wire fence suggested between the roads would not stop a motor-car. On the general question, the fact must be faced that it would be necessary to make roads to suit the motor-car traffic and the motor-omnibus; and if certain of the country highways could be provided with a strip of good sound road, it would be an improvement—an improvement that would have to be made some day, whether it was paid for by the ratepayers or by the persons owning the vehicles. It would be a grand thing for the motor-car traffic and for the roads to have a centre piece 3 yards wide on, say, the Great North Road. Such a strip of Val de Travers asphalt could be laid for about £4,500 per mile. The motor-omnibus traffic was increasing in London, and in time it would be the general mode of conveyance throughout the country. When it was remembered that the roadwork and wiring of an electric tramway cost about £9,000 per mile, the advantage of the motor-omnibus would be evident. Nearly every motor-car made was an improvement upon the last; and if such strides had been made during the last 12 months, much might be expected during the next 10 or 15 years. The motor-car was the future vehicle for the country, and, whatever might be the views of municipal councils with regard to municipalizing their tramways, he would not regard any such undertaking as affording gilt-edged security for money invested in it. If the legislature would frame a Bill to tax motor-vehicles, it would be possible to obtain 5 per cent. per annum on the cost of construction, and thus secure a good road for that class of vehicular traffic. He had travelled from Leeds to London in 26 hours by the mail-coach, and now he could do it in $3\frac{3}{4}$ hours; and it was quite evident that the great wealth of the country had been amassed largely by the saving of time. The motor-car would be the chief means of utilizing time and adding to the wealth of the country; and a tax on such vehicles would be welcomed when the money thus raised was applied to the repair and construction of roads. Mr. Fowler.

Sir John Thornycroft.

Sir JOHN I. THORNYCROFT suggested that the dust nuisance might be greatly lessened by allowing the wind to carry away the dust as it was raised. In England, where the roads were frequently bordered by tall hedges and woods, this could not occur; but in the north of France, where the roads were exposed, side winds cleared the dust completely from them. There the lower branches of trees standing near the sides of the roads were cut away in order to allow free movement of air. Mud was produced not only by the grinding of the surface of the road, but by the sliding of one stone upon another to a considerable depth when the road was saturated with water. In France there was generally a ditch on each side—often 5 feet deep—with a considerable width of grass between the ditch and the road, and he had been much struck by the excellence of the roads and the absence of dust in dry weather. The excessive subdivision of the control of English roads led to extravagance and waste; and neighbouring sections were stepped in a manner which was very uncomfortable for anyone travelling at a reasonable speed. Fast driving through villages was prevented in some parts of Germany and France by watercourses across the road, about 9 inches deep and 6 to 8 feet wide, with a steep incline on either side of the watercourse. He recommended the construction of a few such watercourses to those road-authorities who were unable to prevent people from driving too rapidly through villages. There was something to be said in favour of Mr. Blackwall's proposition to divide the road, and perhaps something to be said against it. Roads for light traffic only required little foundation, whereas roads for heavy traffic, such as traction-engines, must have good foundations, otherwise the road was expensive to maintain. If the width of foundation could be lessened in extent, it might perhaps be economical to divide the road as suggested; but he doubted whether any railing was required on the central footpath. It would hardly keep dogs off the road. The camber of the road was a very important matter, particularly in towns. With wood pavements it was customary to make the roads much more convex than was necessary; consequently no one would go near the gutter on a day when the road was slippery, even cyclists riding down the middle of the road. The centre of a road might almost be made with a slight angle so as to give the necessary slope for running off the water, and if the inclination of the road for a third of its width was tolerably uniform there would not be the same tendency for everybody to travel on the middle of the road. In the vicinity of Westminster the roads appeared to be laid with some regard to flatness.

Mr. James C. Inglis observed that there had been a time when most railway-companies were content to have in country districts a goods-shed and a passenger-station; but the number of places where it was safe to continue with such facilities for traffic was becoming fewer. Country people could not always afford the time to take their produce and their small manufactures to market, or to bear the expense of horse-traffic; and consequently railways had been obliged to extend their tentacles much more widely. In recent years the Great Western Railway Company, like other railway-companies, in addition to increasing its stock of carts and horses, had become possessed of about seventy motor-vehicles. That fact alone showed that the condition of the roads leading even to the smallest stations had become more important than it had ever been before. He thought too much stress could not be laid upon the necessity for a more central road-authority than existed at present. In one district which the Great Western Company had selected for development—a fruitful district 12 to 14 miles away from a railway—everything had gone well in the summer-time, but when in the winter the authorities repaired the roads with basalt, it had soon been found that a set of tires, costing £127, could easily be spoilt in little more than a week. The District Council being sub-contractors for the County Council, it was represented to them that the railway-company really could not face such a state of affairs, anxious as they were to serve the district, and the authorities were asked to roll the roads as they laid the metal. The surveyor replied that life-long experience had convinced him that metalling worked in naturally by traffic passing over it produced a much more lasting surface than artificial rolling. The railway-company could not combat that proposition, but they stopped the running of their omnibuses; and the ultimate result was that the County Council took over the maintenance of the roads from the District Council. With regard to French roads, driving from Munich to Calais in a motor-car he had noticed a point which strengthened his view as to the necessity of central control. He endorsed everything Sir John Thornycroft had said on the quality of the French roads—except in the villages. Inquiry had shown that the village roads were bad because they were under village control; and evidently the idea that the smaller the authority the less satisfactory was the maintenance was also held abroad. When in Singapore recently he had been astonished by the scientific manner in which the roads were made by Government engineers. In a climate with a rainfall probably twice that of the British Isles, the surface of the road was almost beyond reproach. The good condition of those roads was due to the use of good material Mr. Inglis.

Mr. Inglis. in a proper manner; there was no mixing of soft material with hard. In another instance the Great Western Railway Company received a petition from a local authority for a motor-car service between two towns, one of which was not connected with a railway. The company, feeling that there was traffic enough to justify such a step, had the road examined, and found there were one or two awkward corners, which they asked should be re-formed. The road-authority refused, but offered to allow the company to do the work. The consequence was that the service was not put on. He believed that in Ireland the roads were better made than in England, because they were looked after by a better class of men. In 1,289 cases the Great Western Railway Company had to maintain short lengths of road-surface, due to embankments or to bridges crossing the railway, and they had initiated the plan of making a contract, wherever possible, with the local authority; they now had no less than 775 contracts in force, for the maintenance of roads so as to secure continuity of surface. He quite agreed with Mr. Jebb as to the necessity for removing the mud from roads; it was a most important point in maintenance.

Mr. Humphreys. Mr. H. Howard Humphreys remarked that he could not claim the credit for the construction of the road in Woolmer Forest alluded to by Colonel Crompton, because, as mentioned by Lord Montagu, it had been built by Mr. W. J. Taylor. Mr. Humphreys had had some connection with it, but comparatively a small one. During the last 5 or 10 years, with the growth of self-propelled traffic, the question of roads had been increasing in importance, and it had now reached a more or less acute stage. For some years past a body of men interested in roads had been trying hard to induce the Government to introduce some measure whereby the number of road-authorities might be reduced—a preliminary step which appeared to be highly desirable, as the number of authorities controlling roads was legion. He believed in England and Wales no less than 1,850 local authorities were concerned with the maintenance of roads: a vast number of methods were employed—and in some cases no method at all. It was hardly fair to say, as was said in Mr. Blackwall's Paper, that the amount spent on county main roads was only about £32 per mile per annum. Vast improvements had taken place in the maintenance of county main roads since the advent of the Local Government Act of 1888, which constituted county councils. From the Local Taxation Returns he noticed that the cost of the maintenance of about 16,000 miles of rural county main roads had risen since 1897 from £55 per mile to £70 per mile in 1903. That in itself might mean nothing; but those who had to do with the inspection of roads

knew that it meant a great deal: he could say unhesitatingly that value had been received for that money, and that the county roads to-day were in a far better condition than they were in the days of the old highway-boards or turnpike-trusts. In considering the present question it was necessary to take a rather broad view. The advent of self-propelled traffic would prove a very important commercial factor, apart from the purely pleasure traffic. At present the light car was looked upon more or less as a rich man's plaything: the bicycle 25 years ago was more or less the plaything of the moderately well-to-do, but to-day the bicycle was a humble though essential factor in commercial life: and he believed that to-morrow the moderately fast car would also be an industrial necessity, and still more so the motor-wagon. At the present time he believed there were only about 1,500 motor-wagons in England; but the potentialities of that vehicle had not been fully realized, and he thought that in another 20 years it would be necessary to go to the British Museum in order to see a dray-horse. The horse was being rapidly displaced, and the rate of progress would not be that of arithmetical progression. The question was whether, with an average expenditure of about £70 per mile per annum on main roads, it was desirable to increase that expenditure—whether the time had not come for some totally fresh legislation with regard to roads. Many people thought that the time had come, and that a Bill should be pressed forward. He believed a Bill was before the House of Commons last Session, forming, as it were, the thin edge of the wedge, with regard to Government control of the main arteries of traffic. A Departmental Committee upon Highway Administration sat in 1903 and their recommendations were extremely good. They advised that exchequer grants should be made towards the county main roads, subject, he believed, to the condition that Government inspectors should have the power of withholding the grant if any section of road was not maintained to their satisfaction, and to a proper standard of strength. He failed to see how there was to be anything like a uniform standard of strength on the main roads of the country unless there was some such jurisdiction. The question of main roads in the country was more or less a national one and should not be left to local councils. A body like the Somersetshire County Council, removed from all progressive influences, considered that if their roads sufficed to carry a load of straw, they had done their duty. They did not consider, as a rule, the question of through traffic, and there was something to be said for their position. They said, "Why should we pay taxes for roads which we care nothing about, so long as they are sufficient for our needs? Why should

Mr. Humphreys.

Mr. Humphreys. not the nation contribute something towards those roads?" It was impossible to answer that argument. The nation should contribute towards the cost of the roads in poor agricultural districts, because they were needed for through traffic. Local expenditure had practically reached a limit. In the county of Kent, for instance, the average cost of main roads a short time ago amounted to more than £160 per mile per annum, and in the county of Middlesex it was still more. In Herefordshire he believed the expenditure was only about £30 to £35 per mile per annum. That seemed to require equalizing to some extent. Everyone was interested in the metropolitan counties, and he thought some of the distant agricultural communities might by indirect taxation bear at any rate a part of the cost that had to be paid by the metropolitan counties. One vital point appeared to have escaped the notice of the Authors, namely, the question of the maintenance of bridges. At the present time there were five or six different authorities engaged in that work—county councils, borough councils, town councils, urban and rural councils, and in some cases lords of the manor—in addition to private bodies such as railway- and canal-companies. It was difficult to find a stretch of road 10 miles in length where the bridges were anything like uniform in strength. Some would carry a load of 20 tons, others must not be loaded with more than 8 tons; and over some no self-propelled traffic of any kind was allowed. How could that state of things be dealt with unless the Government stepped in and to some extent standardized the strength of bridges? A contributory item causing confusion in recently-constructed metal bridges was the fact that nicely-printed lists of steel joists were sent to local councils by the makers, which joists were said to be capable of carrying certain loads under given conditions. Often a misunderstanding occurred in the local surveyor's office, and girders which were calculated originally for distributed loads, were put in for rolling loads, with the result that the factor of safety did not come out quite correctly. Many bridges had been spoilt in that way. Whilst such lists were of the greatest possible service in the hands of those who knew how to use them, they were an absolute danger in the hands of those who did not. He regretted to find himself to some extent at issue with Sir John Thornycroft on the origin of dust. The dust of a road was due to three principal causes. In the first place, when local authorities made roads they were in a great hurry to get their steam-rollers away, and they applied a mixture of ordure, turf, earth, road-scrapings, etc., as a sort of poultice on the surface of the road, ran the steam-roller over it, and rested satisfied with their labours. If the road was

mended in the early spring, by the time the summer heat came along the mixture was dry and loose, the stones came out and rubbed one against another, and the road was destroyed. If the road was made in the early autumn it remained sodden, with the wet poultice on it, and when the frost came all the loose stuff was expanded, the stones raised, and the road destroyed. Instead of applying palliatives it was necessary to get at the root of the trouble and to make roads on the macadam principle, by packing every ounce of solid stone possible into the road-surface. Another fruitful cause of dust was particularly in evidence in the western counties. If a line were drawn from Flamborough Head, through Rugby to Axminster, the whole of the south-eastern portion of the country was what might be called a soft-stone country, while the north-western portion was the hard-stone country, where there were various intrusions of trap and basaltic rock, and it was comparatively easy to get a thoroughly good road-stone. Local authorities in the west of England did not appear to recognize when the economical limit of the local road-stone was reached; and consequently they insisted on piling on soft or comparatively soft stone, although, as they well knew, self-propelled traffic had come to stay. The result was that the limestone roads of, say, Somersetshire were an abomination either in summer or in winter. Limestone roads were an abomination in every way to ordinary traffic, and above all to rubber-tired vehicles, because more side-slip occurred on a damp limestone road than on any other kind of road. No change would be brought about unless the local authorities could be made to recognize when the economical limit of local stone was reached. The camber shown by Mr. Blackwall was utterly impracticable for motor-traffic, because in self-propelled vehicles the width of wheels had been greatly increased and with excessive camber one-half of a wide wheel would not have any contact with the road-surface at all. On the other hand, with a very flat camber—sufficient to draw the water off, which was all that was required—the traffic was distributed over the whole surface of the road, and every part of the wheel-tire was in contact with as large a surface as possible. In conclusion, he hoped the Institution would throw its whole weight into the scale in favour of good roads in this country, and of centralizing the control. He was certain that what applied to railways applied also to roads, namely, that if the country would make the roads, the roads would make the country. Mr. Humphreys.

Mr. H. T. Wakelam thought the plea put forward by Mr. Jebb was deserving of sincere sympathy; but there was a practical side to the question as well as a theoretical one, and that practical side might be summed up shortly in the word "money." Certain arguments Mr. Wakelam.

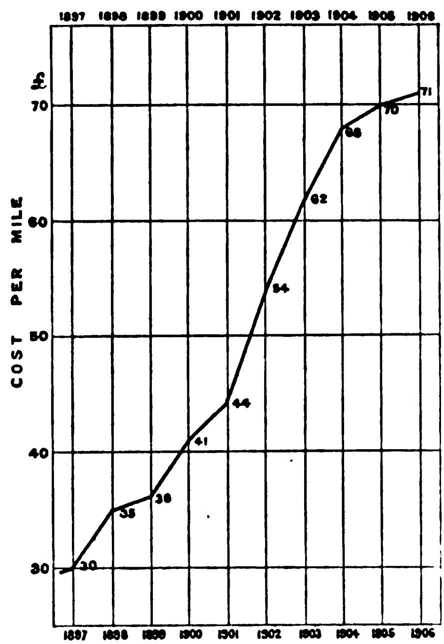
Mr. Wakelam. had been put forward which he could not accept. Colonel Crompton's view that the road formed part of the machine might be correct as regarded railways, but the cases of roads and railways were not analogous. The railway-company owned both road and machine, whereas the road and the motor-car were the property of different owners. If the conditions were the same as on railways, the driver of a motor-car who lost a tire would have a right to demand another from the road-authority. The experiments carried out in Kent had been anticipated by about 12 months in Middlesex, where experiments were being carefully conducted with a view to find the most satisfactory material for use on the Middlesex roads, which carried heavy traffic. No doubt more scavenging would help to get rid of much of the dust nuisance, but the whole question was one of money, and it was impossible for rural councils and county councils of agricultural counties to find the funds to carry out extra scavenging. Mr. Fowler's remarks as to night watering of roads were applicable only to town roads, because rural roads were not even watered in the daytime, and for the same reason—want of money. Mr. Jebb's suggestion that soft material might be mixed with hard material as a means of saving expense, was against all practice in modern road-making. The two materials wore very unequally, and a mixture of them would be of no value. With regard to waterproof roads, Mr. Wakelam had found that basalt and blue mountain limestone chippings in a bituminous matrix was the most serviceable material that could be used. It could not from present practice be accepted that "tarmac," tar-macadam, tar-paving, "westrumite" and other products of enterprising people were the things which would solve the difficulty. Until the Government granted county councils or district councils money towards the maintenance of the roads, as they did before the advent of county councils, the roads would always remain in their present state. The counties could not afford to improve the roads to satisfy the requirements of the fast motor-traffic. In connection with the dust nuisance, he had taken careful note of the roads over which he had travelled in his motor-car at various speeds, and he had found that a car running at 12 to 15 miles per hour raised little more dust than a horse and trap travelling at the same rate; but at 18 to 25 miles per hour the dust from the motor-car became practically unbearable. At the present time the population of England and Wales was about 33 millions, and there was about one motor-car to every 1,200 people, and one motor-bicycle to every 1,100 people. Some time ago he went into the cost of strengthening the rural roads throughout England and Wales, and found that at a very moderate estimate indeed it would cost no less than £80,000,000 to strengthen the rural

main roads sufficiently to carry heavy motor-car traffic; the district roads, outside the main roads, would cost no less than £184,000,000 to provide a surface suitable for the traffic; and the district roads in urban districts alone would cost £22,000,000. In Middlesex in the last 3 years a third class of road had been instituted. Formerly there had been main roads and first-class subsidized roads, and now there were also subsidized roads of the second class. He did not altogether agree with Mr. Blackwall that the experiments on roads had proved abortive, because during the last 2 years several counties had published the results of experiments which to some extent had been very favourable. But want of money underlay the whole matter, and money was necessary for experimenting; and even should any experiment with waterproofing material prove satisfactory as a whole, the difficulty arose as to how the money was to be raised to carry out the renewal of the road-surfaces of the country, with the object of improving them. Mr. Blackwall proposed that his light road should be constructed without retaining-walls; but as the depressions of a road were not all on the same side, it was impossible to scheme any light road so as to do away entirely with retaining-walls on some parts. The use of large-sized metalling, which was also advocated by Mr. Blackwall, would be against all modern principles. The camber shown in Mr. Blackwall's *Fig. 1* was probably exaggerated to show his point more clearly. On the question of centralizing the control of roads, Mr. Wakelam's position as an engineer to a county council precluded his saying whether or not the proper thing to do was to put all the roads into the hands of the county councils. He thought the Institution should bring the requirements of the country to the notice of the Government and help to obtain from the Government as large a grant as possible. Mr. Wakelam.

Mr. W. J. Taylor agreed entirely with Mr. Humphreys and Mr. Wakelam in regard to State aid, or partial State control. He believed that all the questions which had been raised as to the methods to be adopted for improving roads for the newer forms of high-speed traffic would be readily answered by those engineers who had had experience in road-maintenance, if only funds could be found for the work. The Hampshire roads afforded two examples of the difficulties in meeting the cost in connection with maintenance and strengthening. *Fig. 1* (p. 32) showed the growth of expenditure and *Fig. 2* (p. 33) the growth of the quantities of material used per mile on the Hampshire rural roads. In the first year shown (in *Fig. 2*, 1898) nearly the whole of the material was flint and gravel. The County Council realized the increasing demands on the roads, and endeavoured to strengthen them by putting on more flint and gravel, rising to the maximum in 1903, to prepare them for the future Mr. Taylor.

Mr. Taylor. coatings of imported material which were now being applied. The result had been that during that short period the expenditure on the roads had been doubled, rising to £71 per mile in 1906, which, from the ratepayer's point of view, was a very considerable advance; and a great improvement in the roads had been the result. There was still a considerable gap between their present condition and the condition in which it seemed to be considered that they ought to be, in order to cope with the developments of modern traffic; but in the matter of expenditure

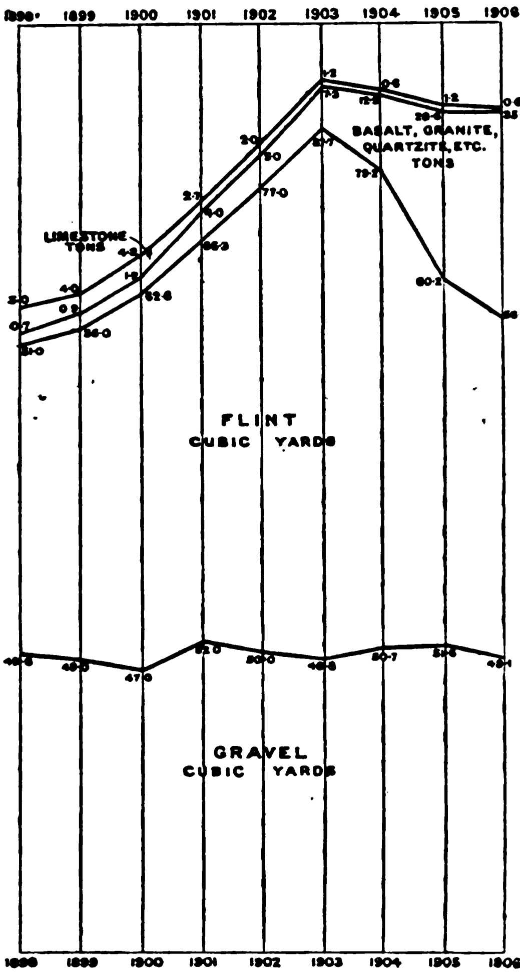
Fig. 1.



the Hampshire County Council had gone as far as they could, and without State help it was almost impossible to do more. The road in Woolmer Forest, to which reference had been made, extended from Farnham southward towards Petersfield. The damaged portion was 10½ miles in length, ran over greensand and gault, and had cost about £44 per mile to maintain up to 1901. In that year the Government decided to erect in the vicinity military camps for the occupation of troops returning from the war; and without any warning they concentrated traction-engine traffic upon the road (with material for the construction of the camps) and destroyed it. The road became

considerably depressed in the centre, and the sides were driven Mr. Taylor. half-way up the banks. It was immediately taken in hand by the County Council, on his advice, on the lines shown in *Fig. 3.* The ruts were first filled with limestone, the road was then lined out

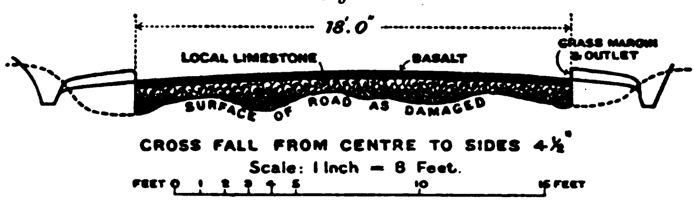
Fig. 2.



18 feet in width, the margins were made up with grass turf, and a further coating of limestone 8 or 10 inches thick was put on and well packed. It was hoped that the difficulty was being overcome, especially as the Government assured the Council that the extraordinary

Mr. Taylor. traffic had ceased. Efforts were concentrated on other portions of the road, but there was a recurrence of the traffic on the first portion, and 3 miles of that road, costing £2,000 per mile, was almost utterly destroyed. It was necessary to begin again and work up until there was a thickness of 9 or 10 inches of limestone covered with 4 inches of English and German basalt. It was found cheaper at times to get basalt from Germany than from England; it was a material of high density, and served its purpose well. His reason for calling attention to the diagram was to explain the enormous cost which would be incurred by county councils if they attempted to strengthen the roads in this way to make them suitable for modern traffic. To treat thus all the roads in Hampshire would cost about £2,000,000.

Fig. 3.



The reconstruction of the Woolmer Forest road had been carried out by the County Council's own roadmen, and had involved the use of 14,000 tons of flint and gravel, 23,000 tons of limestone, 600 tons of quartzite, and 15,000 tons of basalt; and the total cost when the work was completed would be about £38,500. A great deal of that had been wasted by the extraordinary traffic; but the fact remained that to improve the roads by putting in a proper foundation and giving a surface entirely fit for all the developments of high-speed and modern traffic was quite beyond the powers of an ordinary road-authority; and as regarded such remaking and strengthening, unless the State helped, the roads would have to remain much as they were.

Mr. Thomas Mr. R. J. Thomas pointed out that, according to figures he had extracted from the Local Government Board Returns, the increase in cost of 23,769 miles of rural main roads between 1901 and 1904 had been £272,000, or an increase of 17½ per cent.; and on all rural roads the increase was £488,000. It should be borne in mind that the state of agriculture in England, and the state of trade in the smaller rural counties, was pitiful; and if the roads had to be improved as they should be improved for the coming traffic, funds must be found elsewhere. County councils had been trying to do their best, and had increased their expenditure by 17½ per cent. In Buckinghamshire some "tarmac" had been laid for the

past 2½ years, and it had answered very well. The price worked out at 2*s*. per square yard, which on a road 10 yards wide would cost £1,760 per mile. The cost of maintenance in Buckinghamshire for rural main roads was £62 per mile. Another method of temporarily doing away with dust was to tar-paint the road. He had experimented largely with that, and had found that it could be done at 1*d*. per square yard, which, on a road 7 yards wide, would amount to £51 per mile. So successful had the method been, that many residents on roads suffering from dust had paid the cost of tarring. With regard to the construction of roads, it was obvious that mixing soft material with hard was bound to be a failure. It was unfortunately a fact that on heavily-used main roads, however carefully the best granite was put down, heavy fast pneumatic-tired motor-cars abstracted a large amount of small particles of stone from the surface, so admitting water into the foundation. The motor-cars running on the Bath Road during a dry week-end picked out millions of small stones ⅛ inch to ½ inch in size, although the road had been made of the best basalt, without any addition of foreign matter. That difficulty had to be met either by tarring the road, or by using "tarmac." Scraping roads was highly undesirable. When a road had mud upon it the surface was tender, and to use scrapers, particularly horse-drawn scrapers, on such a road was as bad as shaving a man suffering from smallpox. The way to remove mud was to sweep the road well and often. Reference had been made to traction-engines, and he might draw attention to a strange anomaly in traction-engine law. A driver of a traction-engine weighing anything less than 5 tons unloaded, and 12 tons loaded ("heavy motor-car"), was required to have a licence. If he offended three times, his licence might be suspended for life; but next morning he could drive a 14-ton traction-engine, and snap his fingers at all the licences in the world. He required a licence for a 5-ton engine, but not for a 14-ton engine. Mr. Thomas.

Mr. Douglas Mackenzie, as the engineer in charge of the experiments about to be carried out in the county of Kent, pointed out that these experiments related entirely to country roads. Many road-surveyors had endeavoured to solve the dust problem by experimenting, and hundreds of experiments had been conducted in various parts of the country by different men under different conditions; but, with one or two exceptions, it was almost impossible to compare the results. If anything of national value was to be done, it was necessary that the experiments should be conducted with different materials on a section of road where the traffic was identical, and where the conditions as to subsoil, gradient, and other things were such that the comparison might be quite fair. He was glad to say Mr. Mackenzie.

Mr. Mackenzie. that the County Surveyor of Kent and the Highways Committee had been good enough to place a section of road in the county at the disposal of the experimenters—a road subjected to every kind of traffic, and perhaps to rather heavier traffic than the average of the whole county. It was proposed to divide the road into suitable lengths, and to lay these lengths with different materials so that they could be accurately compared. It would then be possible to judge how the various sections of road behaved under similar conditions. At either end of the experimental section were lengths of road which had been re-coated during the previous autumn by the Kent County Council with Norwegian and Cornish granite, and one length had been tar-painted, so that the experimental materials could be compared with the ordinary granite road and with granite road tar-painted. It was hoped to obtain thus data with regard to the cost of laying, and with regard to the life of particular materials. He was sure the results would prove that the cost of maintenance could be reduced by using dustless material. The cost might be doubled in the first instance, but if the materials lasted three times as long, there was an ultimate economy. By dividing the roads of a county, a surveyor could gradually re-coat them without increasing the expenditure; and in the course of 5, 7, or 10 years, all the main roads of the county would be coated with dustless material, and after that the maintenance would cost considerably less than at present. The rapidly-increasing cost of country roads was due largely to the use of unsuitable materials, and also to the fact that the roads were not waterproof. Water soaked through to the foundation, and allowed the subsoil to work through between the stones and form dust and mud; it also loosened the binding and allowed the stones themselves to rub one against another and disintegrate rapidly.

Mr. Aveling. Mr. T. C. Aveling thought that questions of materials and methods would settle themselves, the main point for discussion being the anomaly of so many different authorities endeavouring to carry out similar work. The only way to get over the difficulty would be to standardize the roads into first-, second-, and third-class, and to have a settled specification for each of them, to which all the road-making authorities of the country could work. As an instance of the kind of organization required, he might mention that the Paris-Madrid motor-car race took place on a Sunday, and when returning over the route on the following Wednesday he found that over the whole 375 miles the dust and stones had been swept back to the crown of the road. The question of cost to an engineer was always an important matter, but he did not think there was need for any anxiety about cost, since the benefits of good roads would be very great.

Mr. ALAN A. G. MALET, as an Inspector of the Local Government Board, supported what had been said with regard to the necessity of centralization and organization. The necessity for large expenditure on the improvement of roads was undoubted, and perhaps some saving could be effected by improved organization. It might perhaps be advisable in some cases that local authorities should insist on their officials in charge of roads visiting other localities, and should pay expenses incurred in doing so. The only way to learn was to go and see what other people were doing in other places. A new invention was coming forward, the "pedrail," which, if it proved successful, might have an influence on the design of roads. He had been deputed by the Local Government Board to accompany a "pedrail" engine on a journey from Lincoln to London; and if the invention were a mechanical success he would not be surprised if all the authorities in this country responsible for road-construction combined to bring forward a Bill requiring heavy wagons to be mounted on pedrails. The experiment in question had been carried out at the instance of the War Office, and during preliminary trials at Lincoln the pedrail engine had been taken over soft cinder roads, which were hardly affected at all by the pedrail. A new design had now been brought out which, it was stated, had entirely removed the mechanical difficulties; the diameter of each "foot" had been altered from 11 inches to 15½ inches, and the pressure per square inch had been reduced, in spite of a much heavier engine, from 38 lbs. to 22 lbs. per square inch on the road-surface. Mr. Malet.

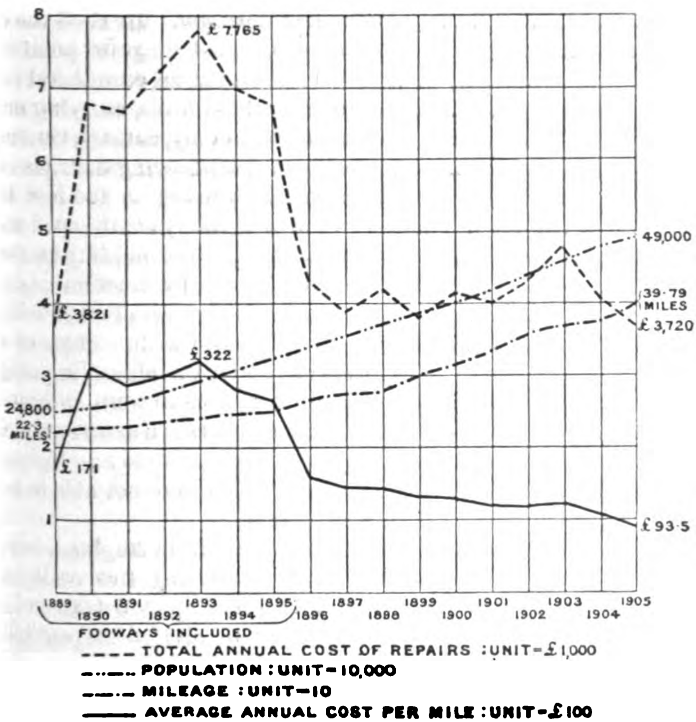
Mr. R. J. G. READ, as a Somersetshire man, objected to the criticism which had been passed upon the roads of his county by a previous speaker, who must have travelled along some of the by-lanes. It was true that the roads generally were of limestone, and when he was a boy they were repaired from the local quarries, generally consisting of limestone from the oolite formation or the white and blue lias. The limestones of the lower oolite were comparatively soft, and therefore the roads soon became muddy. But now throughout the county the stone used was mountain limestone, mainly from the Mendip Hill district. It was much harder and made better roads; and he considered that many of the main roads were very good roads for traffic. A further improvement had been brought about owing to the discovery in the Mendip Hills of intrusions of a volcanic nature, forming whinstone and volcanic trachyte; and whinstone was now being supplied for the roads instead of mountain limestone. It was true the district roads left much to be desired in the matter of straightness, but the authorities could not afford to improve them unless State help was given. Mr. Read.

Mr. Smith. Mr. E. SHRAPNELL SMITH desired to say a few words from the users' point of view. Better country roads would be welcomed by all users of omnibuses and heavy motor-wagons, but a knowledge of the state of the foundations of the road any vehicle was going to run upon would also be very useful. Many a user would not run upon some roads had he permission to sink trial-holes, under the supervision of the surveyor, at his own expense, and subject to such conditions as the local authorities imposed, which would inform him as to the state of the foundations of that highway. Another point not always appreciated by road-engineers was the great reduction in the intensity of weight per inch of tread that had been brought about in motor-vehicle construction, owing to the increase in tire-width allowed by the Heavy Motor Car Order of 1904. Following the different trials with such vehicles, which it had been his privilege to conduct, an experiment had been made in Lancashire with a number of the vehicles, which sometimes caused a pressure of 13½ cwt. per inch width of tire on a 3-foot wheel, with the result that often two-thirds of the vehicles were in difficulties on the road, at no small expense to the owners and to the county. But while the average for the whole country of the weight per inch width of tire of a motor-wagon was about 10 cwt. 2 years ago, to-day it was only about 6 cwt. Before that Order came into force, if the tires were widened enough to protect the roads, the machine was rendered illegal by its excessive weight in the unladen condition. That the cost of the road was the key to the situation was appreciated by the user as much as by anybody, and the necessity for through traffic seemed to point in the direction of some assistance from the Imperial Exchequer. Colonel Crompton and himself, in evidence before the Motor-Car Commission, had been authorized by the Motor Van and Wagon Users' Association, after very full deliberation, to say that, as a step of expediency, having regard to the desirability of assisting the development of the traffic, the users were prepared to accept a reasonable annual charge per ton of axle-load upon their machines. Of course, that would not be a very large contribution; but the proposal at least showed that those who were responsible for the introduction of these vehicles recognized the difficulty. The question of the use of omnibuses and motor-wagons was of greater importance to England than to any other civilized country, not because the aggregate length of the roads was greater, but because the mileage of roads per unit of area was greater, and the centres of population and industry were closer together.

Mr. Cooper. Mr. C. H. COOPER observed that the effect of keeping a road dry was well shown by the state of the Surrey roads in a dry summer. In years gone by, the Guildford road, after leaving Surbiton, was

practically a pebbly beach in dry weather; for until Guildford was reached nothing was done in the way of watering to bind the road together. In a Paper read at the St. Louis Engineering Congress, 1904, macadam roads and ordinary ballast roads were termed "water-bond" roads; because to keep the material together a certain amount of moisture was necessary. On the previous Sunday he had travelled over a considerable portion of the branch of the Brighton road that ran through Sutton, and also over the Epsom road, and he Mr. Cooper.

Fig. 4.



had found the surface quite smooth, due to the passage of motor-cars over it. The motor-cars numbered at least four for every horse-drawn vehicle. Fifteen years ago there would have been a smooth track along one portion where bicycles travelled, the rest of the road being partly dusty and partly muddy: nowadays the motor-wheel with its india-rubber tire seemed to distribute moisture evenly over the road. In order to avoid damage from heavy traction it was necessary to get rid of the iron tire and to have a tire as nearly approaching an india-rubber

Mr. Cooper. tire as possible, as that was the kind of tire which carried the maximum load on an ordinary road with a minimum of damage. He could not understand how Mr. Jebb could advocate a mixture of limestone with other material for road-surfaces. Anyone who had had to do with macadam or ballast roads knew that, in order to get economical roads, uniformity of material was essential in the top layer, and if it could be obtained in the foundations, so much the better. With any mixture, the softer material was constantly being worn out, and the road became expensive to maintain. *Fig. 4* showed the cost of the roads of Wimbledon during the last 16 years, not taking into account scavenging and watering. In 1905 the cost had been reduced to about £93 per mile. That diagram pointed to the fact that uniformity of material resulted in an economical road, because during the whole of the 16 years those roads, carrying much traffic, had been maintained with one material only, namely, Guernsey granite from St. Samson; while roads in neighbouring districts were maintained with flints. The diagram also pointed to the fact that motor-traffic did not add to the cost of a properly constructed road, for during the last 6 years at least one-third of the building-material used in the district had been brought into it by traction-engines. With regard to highway-bridges, the condemnation of such bridges by local surveyors was a difficult matter to deal with. Most of the bridges throughout the country were old, and it was almost impossible to say what they would bear. Many iron bridges were so covered with material that it was impossible to examine the structure properly. He questioned whether Mr. Humphreys could point to any highway-bridges constructed in the last 15 years which were not able to bear a moving load of 15 tons, except on by-ways.

Mr. List. Mr. G. H. List had never had charge of roads in England, but he had had charge of traffic over roads traversing five contiguous counties, for a period of 4½ years, with loads up to 6 tons on four wheels; and his experience of the effects of divided responsibility on the maintenance of roads, as affecting users of roads with extraordinary traffic, might be interesting. In two counties no trouble about the roads had occurred. In another county 14,500 tons had been carried over various roads without any preliminary agreement, and the payment had been settled amicably at 2¼*d.* per ton-mile. In the same county 30,400 tons had been carried over rural district roads, and, by agreement, the difference between the actual cost of maintenance and the average cost for the previous 3 years had been paid, the charge averaging 1¾*d.* per ton-mile. In another county 7,000 tons had been carried over a main county-council road which had just been remetalled and steam-rolled, and nothing

was charged, the surveyor saying that the traffic had done the roads Mr. List.
good. In the fifth county interminable trouble had arisen. First one road, over which most of the traction-engine traffic went, was taken over. It was in very bad condition, and was rebuilt with the best granite, steam-rolled. But that did not satisfy the authorities, and in the second year an agreement was made to pay the difference between actual cost of maintenance and the average cost during the preceding 3 years. That, too, did not satisfy the authorities, who, for the remainder of the time, sent in their account of the estimated cost of maintaining the roads, which totalled 1*s.* 1*d.* per ton-mile. In the same county similar traffic had been carried over rural district roads, without any agreement with the authorities, at a charge of 2*d.* per ton-mile. The differences in different counties under the same circumstances were therefore extraordinary. Seeing how such differences affected users of roads who were not residents or ratepayers, and had no remedy, he was in agreement with previous speakers in advocating some central control of roads. This need not disturb or displace present holders of executive charge, but would ensure a uniform system over large areas of similar nature. He would not advise one body for the whole kingdom, but would divide the country into provinces, districts, or circles, suitable to the local conditions prevailing. As an instance of the mischief that had resulted from divided responsibility, he might point to the present condition of the canal system in England, for which however there was now some hope of remedy. Had the trunk railways, originally many separate lines, been left to this system of divided responsibility, there would have been confusion indeed. He had had charge for 7 years of Indian roads which were managed by a central authority in each Province. The roads were of three classes:—First-class, raised, bridged and metalled; second-class, raised and bridged, but not metalled; third-class, neither raised nor metalled, but with occasional bridges over important streams. Roads of the first class were imperial over the whole Province; those of the second class were provincial, but uniform in each district (county); while roads of the third class were purely local and under the control of each rural or parochial authority, as it might be called. In each district (county) there was a district engineer, directly responsible to central and local authorities for the maintenance of roads of the first and the second class, but only called in as adviser in the case of third-class roads. The tenor of the discussion led him to think that the adoption of some such system in England would lead to economy and efficiency. He did not at all like Mr. Blackwall's proposal for twin roads as shown in his *Fig. 1*, and thought that the central footpath and

Mr. List. wire fence would be traps instead of safeguards; indeed, Mr. Blackwall was far too sanguine as to this plan keeping off children and dogs. Mr. List would widen the roads where possible, and make strong side-tracks for up and down heavy slow traffic, with a lighter centre for a faster light traffic. This would separate the traffic sufficiently, allow crossing without special arrangements, and facilitate repairs and maintenance. In his experience in India this plan of working the wheeled traffic was carried out on second- and third-class roads; and although first-class roads were not metalled for this system, the slow vehicles drawn by bullocks automatically adopted it, so that the bullocks' feet did not suffer so much on the softer unmetalled margins of the roads. Before the advent of mechanically-propelled traffic, especially heavy motors, the roads had been sufficient for all purposes: they must now be strengthened to meet new conditions, and funds must be found. But it was hard on the ratepayers of poor districts to have to make and keep their roads to a standard beyond that required for their own needs. He thought the "foreign" users should be made to contribute by a sliding-scale tax or licence on motors, based on weight, power, and speed. He would place no limit on speed, except through towns and villages, and at dangerous places, to be specially signalled. But anyone who wanted to run at higher speeds than 20 miles per hour on country roads should be made to pay handsomely for the privilege.

Mr. Shelford. Mr. Fred. Shelford remarked that Mr. Blackwall's proposition to have a heavy road on one side and a light road on the other did not appeal to him as a motorist. The main thing required by everybody who used a road was width, and Mr. Blackwall's proposal was tantamount to putting a footpath and fence down the middle of a road wide enough to enable it to be done. There would also be confusion at cross roads, because in turning off the main road on to a cross road, the division of the road on which the vehicle had to travel came to a different hand to the driver, according to which road he took. Wherever there was a gate leading into a field, or a branch road, or a cross road, and also at the bottoms and the tops of hills, the footpath must cease, and the foot-passenger must go into the road among the traffic. The fence was extremely light, and in the dark a motor-car might easily knock away a length of it without knowing anything about it. The improvement of roads generally was a question of cost. A great deal could be done for the £1,800 per mile mentioned by Mr. Blackwall, without adopting his scheme. The footpath should be placed at the side. A foundation of hand-pitched stone on edge, to carry the heavy traffic, and a

surface of asphalt, or "tarmac," or similar material, should give an admirable road. He agreed with Mr. Jebb's suggestion to put all roads under the county councils, which would undoubtedly be a good plan. Motorists at present met with many obstructions on roads, *e.g.*, live stock and children; and the public needed to learn that the roads were no more safe for live stock than was a main line of railway. Mr. Shelford.

Mr. C. W. S. Crawley described an experiment made by him to test the actual effect of pneumatic tires on the surface of roads. In the summer of 1905 he was staying at Sunbury, where, on the road between Sunbury and Staines, the motor-traffic was exceptional; in fact, on Sundays between 11 and 1 o'clock there was a stream of motor-car traffic with hardly any horse-traffic at all. Frequently he had counted more than twenty motors for one horse-drawn vehicle, and he thought the motor-traffic might safely be put at 90 per cent. of the whole. One part of the road was covered with large, coarse grit and small stones, a fact which had been pointed out to him by a friend as showing how much the motors destroyed the roads. He thought over the matter, and next time he visited Sunbury he took with him a fan. At 1 o'clock he went out to a spot where the road did not show any grit, just off the track of the motor-cars, which kept to the middle of the road. He then gently trod down a few square feet of the surface, to loosen what had been caked by the previous night's dew, and thoroughly fanned it; after 2 or 3 minutes all the dust had gone, and there was just as much grit and small stones as was to be seen on that part of the road over which the motor-traffic had passed. He did not wish to dogmatize from one case on one road; but he would like road-surveyors to try the same experiment, and if they got the same result, they would probably agree that instead of damaging the roads motors were very efficient means of cleaning them. Sweeping or scraping a road removed both good and bad material, but the motor-car removed the bad and left the good behind—he was speaking, of course, from the point of view of the road-surveyor and not from that of the neighbours. He believed motorists fully appreciated the manner in which road-surveyors were dealing with the subject. They seemed to have grasped the fact that motors had come to stay, and they meant to make the roads fit to carry them. The motorists on their side were only too glad to meet the surveyors halfway. Some years ago, very careful tests were carried out at the Crystal Palace by the Automobile Club; and a permanent Dust Committee had been appointed. The first work had been done chiefly by Colonel Crompton, Mr. Cooper, and himself, and it had come home to them very strongly that there were cars which were Mr. Crawley.

Mr. Crawley. very good and others which were very bad in respect of raising dust. After those trials Colonel Crompton and he had laid down what they thought were probably the lines of a fairly good dustless car, and some makers had been building closely on those lines. Some of those cars had been tested at the last dust trials. Mr. Hawkins provided a very dusty road, which had had several batteries of artillery over it on the previous day. Over the bad parts of that road the specially designed cars had raised very little dust compared with the others, though of course any car would be a nuisance on such a road. On coming to a good patch they had raised very little dust at all, although in some tests they had travelled at the rate of 40 miles per hour—by special permission.

Mr. Hawkins. Mr. J. F. Hawkins, speaking on the general question of the maintenance of the roads as a whole, compared his own county of Berkshire—a poor agricultural county to which a grant would be of great service—with Cheshire, a very rich county. A 1*d*. rate in Cheshire brought in £13,766, while a 1*d*. rate in Berkshire brought in £4,867. His Council were doing their best to improve the roads in Berkshire, but it was uphill work. In Cheshire it was possible to get the finest macadam in the world delivered anywhere in the county for about 10*s*. per ton, but in Berkshire it was not possible to get macadam which was any good at all for less than 13*s*. per ton as an average price. There was a very large amount of through traffic in Berkshire. If the county councils had taken over all the roads when the Act came into force, he was sure the roads would now have been better than they were. The smaller authorities had in every case to study economy. For instance, last summer he saw a district surveyor working a 15½-ton roller on a by-road. Under those conditions the macadam was simply being crushed out of sight; he therefore asked why it was done, and the surveyor's reply was that he had to keep down the expense as much as possible, and as he was only allowed a certain amount for the rolling, he had to get the steam-rolling done quickly. Drainage seemed to be the last thing thought of on by-roads. With regard to camber, he had recently had to remake a piece of the Bath Road, and after removing about 9 inches of gravel which had been put on since the old quarter-sessions gave up the roads, he came across a perfect macadamized road, with 3 inches of hard broken boulders—as good a road as could be wished for. The amount of camber on the road was absurd; it was only possible to travel on the middle. The camber shown on Mr. Blackwall's diagram was wrong, because the wheels of a car would never be level and would be always cutting the road. Along the old Bath Road there were pumps at intervals,

Mr. Hawkins.

and he understood that in former days the roads were watered from the pumps, although he could not get any definite information. He had had one of the wells examined, and had found it to be 60 feet deep, the brickwork extending down for more than 30 feet. The pumps were all out of order, and would cost a large amount of money to repair. With regard to "tarmac," he found the great difficulty was that when the material arrived, especially in the summer, the tar was found to have sunk to the bottom of the truck during transit by rail. When the tarmac was put on the road it was impossible to get it properly mixed again, without re-heating; and where there was too much tar it was affected by the weather and would not settle or become solid; in fact, in hot weather it was on the spew the whole time. He would be glad to know how to overcome these difficulties.

Sir John Wolfe Barry.

Sir JOHN WOLFE BARRY, K.C.B., Past-President, thought that no worse plan of laying out a main road than that proposed by Mr. Blackwall in *Fig. 1* could be devised. He had some experience of driving about in the neighbourhood of London, and he had suffered acutely from the practice of putting long rows of pillars in the middle of good roads, in order to carry overhead conductors for the electric trams. How any person could have adopted such a method of injuring existing public roads passed his comprehension; but even that was better than dividing a fairly wide road into two 21-foot roads by a continuous longitudinal footpath between them; and he had no hesitation in saying that a road 42 feet wide would carry, with less inconvenience, at least twice as much traffic as two separate roads each 21 feet wide. His own view was that there should be as few gutters and as little camber as possible. Mr. Blackwall's plan put the foot-passengers where they did not want to be; they desired of course to be at the sides of the road, not in the middle, where they would have to cross the traffic whenever they wished to get to houses or property at the sides. Besides that, the number of gutters would be doubled, and therefore the inconvenience to traffic which must arise in connection with gutters, whether the camber of the road was large or small, was doubled. The whole proposal seemed to be the worst arrangement he could imagine for laying out roads. He would be glad to hear whether disk wheels had been tried for motor-cars. He had often been struck by the considerable quantity of dust stirred up by the rapid rotation of the wheel-spokes, and he thought disk wheels were quite worth careful trial, if it had not already been done.

Mr. Swinton.

Mr. A. A. CAMPBELL SWINTON wished to emphasize Colonel Crompton's point that the road and the vehicle should properly be considered as one machine for the transport of passengers and goods. The statement that any particular form of road-construction was

Mr. Swinton. too expensive, had, by itself, no point. What had to be considered was the total expense of the wear on the vehicles and their tires, and the maintenance generally of both them and the road. What would be thought of a railway-company which used inferior rails because they were cheap, although they might lead to great expense with the rolling stock; or, conversely, of a railway-company which used cheap rolling stock that played havoc with the rails? How the expense was to be apportioned was another matter: that was scarcely for consideration by the Institution, but by Parliament and other authorities. The question to be discussed at the Institution was really what would produce the greatest ultimate total economy, both for the users of the roads and for the persons who paid for their making and mending. He had often noticed of late years, since rubber tires came into use, portions of country roads with vast numbers of loose stones lying on them; and he suggested to landed proprietors and others who owned motor-cars that it would pay them to incur the expense of having all the loose stones picked off the roads. The expense would be returned over and over again in the saving of wear on tires. What he was about to say applied chiefly to towns, although in some cases it applied to the country also. It was a leading principle in all machine-design to arrange that wear should be taken up by the parts which were most easily and economically replaced. But in towns horses were shod with iron, wheels were covered with iron tires, and it was arranged that all the wear should take place on the road. A town road was expensive to repair, especially taking into account the dislocation of traffic and the consequent loss. Therefore in a town the wear should be taken up as far as possible by the wheels of vehicles. It should be made penal to use iron tires, or at any rate they should be so heavily taxed that eventually they would disappear. Then the roads in towns might be made of very hard material, perhaps even of cast iron. Such roads would be practically indestructible, and there would be a great saving in never taking them up for repairs. He might have put his point in an extreme way, but he thought the matter was worth consideration.

Right Hon. Sir John Dorington. The Right Hon. Sir John Dorington, Bart., M.P., asked how Mr. Swinton would enforce upon all users of roads the payment of their due share of the cost of maintenance. A proposal in Parliament a few years ago for a wheel-and-van tax had been so extraordinarily unpopular that it could not succeed; and if a Bill were introduced into Parliament enforcing india-rubber tires it would meet with such an amount of obstruction that it could not possibly be passed. Was it quite certain that india-rubber tires were better for the roads than iron tires? Had they not a special

power of disintegration which iron tires did not possess? Much had been said by previous speakers about the disintegration of roads, but it was important to know what class of roads was referred to. Flint roads were admirable in moderately wet weather, but in dry weather, owing to their tendency to disintegrate, they became the worst possible roads, and certainly the most dusty. As Chairman of the Gloucestershire County Council he was much interested in the economical maintenance of roads, and he thought it had been proved that the better the material, the cheaper the road. The best material in Gloucestershire cost about 14*s.* 6*d.* per ton on the roads. Other material could be obtained at 3*s.* 6*d.* per ton, but this was much more costly than the 14*s.* 6*d.* material. A great many materials had been tried in Gloucestershire,—Penmaenmawr, Guernsey, and Clee Hill granite, etc. It was a question of transit which was the cheaper, Penmaenmawr or Clee Hill granite. Wherever the stone could be obtained by rail the Clee Hill carried the day, but wherever it had to be obtained by water the Penmaenmawr granite was the more economical of the two. With regard to the number of authorities managing roads, he thought a great mistake had been made in setting up rural district councils as highway-authorities, in addition to county councils. County authorities ought to have been given at that time power to take over all the roads they pleased; and those not taken over should have been left to the parishes, because they were trifling roads in which the public had no interest whatever, such as a road to a farmhouse, and so on. Now there were two expensive organizations, one nearly as complete in its structure as the other, the county organization and the district organization, with separate surveyors going over the same districts, crossing one another incessantly, and practically travelling twice as far as was necessary in order to look after the roads. He thought one of the greatest improvements to be made would be the transfer to the county of all the rural district roads which the county council chose to take over, abolishing the rural district councils as road-authorities, and leaving the insignificant roads to the parishes. He would not transfer to the parish any road in which the public had any interest; but, as everyone knew, in many counties there were numerous by-paths, scarcely more than green lanes, which were not worthy of the attention of the county surveyor, and which the parish could take charge of without any surveyor at all. Right Hon. Sir John Dorington.

Mr. Jebb considered that there were many serious objections to the scheme for twin roads proposed by Mr. Blackwall, and he felt strongly that no obstruction of any kind should be permitted in any part of a highway, save under exceptional circumstances. Mr. Jebb.

In reply to the discussion on his own Paper, he understood

Mr. Jebb. Colonel Crompton to say that he (Mr. Jebb) thought that many of the difficulties with regard to roads could not be removed without the use of some special kind of stone. All that was said in his Paper was that it was often better to pay a high price for a good stone than a lower price for stone of inferior quality. This opinion had been supported in the discussion, *e.g.*, by Sir John Dorington. He was afraid the concave road suggested by Colonel Crompton would be impracticable for any considerable length; the gratings would become choked after heavy rain and the road would be flooded, and there would be trouble both with the main drain and with the numerous cross drains which would be necessary. He agreed that there was often too much camber on roads. The nuisance caused by mud and dust had been generally admitted. He was glad to have his opinion that sweeping was better than scraping supported by Mr. Thomas, the County Surveyor of Buckinghamshire, who had had large experience in the maintenance of roads. Of course the sweeping should be done during rain or as soon after it as possible. He regretted that he had not made clear what he meant by suggesting that a little limestone might be used with harder macadam. What had been in his mind was that a very small percentage of good limestone, in the form of chippings, might safely be used, and that this would help to bind the harder stones together. He entirely agreed that nothing could be worse than to mix a hard and a soft material together in anything like equal proportions. In the manufacture of "tarmac," with the qualities of which several speakers had been favourably impressed, the iron cinder was now broken and the tar material was mixed with it before the cinder had cooled. He was told that the chief failures had been due to the fact that the stone or cinder had absorbed moisture before it was mixed with the tar. Mr. Fitzmaurice, the Engineer to the London County Council, had kindly given him for publication the appended memorandum of the result of some experiments which he made with tar-paving on the Victoria Embankment in 1905. From this and other reports it seemed to be prudent, for the present, to refrain from expressing any definite opinion on the durability of tarmac or other similar material when subjected to heavy traffic. The experiments made by Mr. Thomas with "tar-paint," as a method of temporarily doing away with dust, were worthy of consideration. The main object which he had had in view in bringing this subject before the Institution was to get, if possible, a strong expression of opinion that the present system under which the roads were maintained was unsatisfactory and wasteful, and that a large number of road-authorities ought to be abolished. This contention had been,

he thought, unanimously supported; no argument had been used in favour of the continuance of the present system. It should not be left to the county councils to take over what roads they choose; they should be compelled to take over and maintain all country roads in which the public had any interest. The question of bridges, raised by Mr. Humphreys, was referred to incidentally in Mr. Jebb's Paper; and in his evidence on behalf of the Canal Association, given before the Royal Commission on Motor-Cars, he had recommended that all bridges carrying roads over canals should be taken over and maintained (of course, on reasonable and equitable terms) by the road-authorities. In advocating the abolition of the district councils as road-authorities, he did not bring any charge of incapacity or want of skill against any district-council surveyor. Some speakers had advocated State control or partial State control of roads. No doubt many good arguments could be used in support of this proposal, but he feared that the large questions which would be raised in considering it would have the effect of shelving the subject for a considerable time. His own proposal would cause the minimum of interference with the existing organizations. Mr. Jebb.

EXPERIMENTAL TAR PAVING, VICTORIA EMBANKMENT, 1905.

NOTE BY MR. MAURICE FITZMAURICE, C.M.G., M. Inst. C.E.

When arranging for the annual re-metalling of the Victoria Embankment, it was thought desirable that an experimental length of tar macadam should be laid down.

The portion of roadway, which on the Embankment is 64 feet wide, selected for the experiment, lies between the Savoy approach and the Cleopatra's needle, and measured 300 feet in length.

This length was divided into three sections of 100 feet in each, with a view of trying different material as an aggregate.

That to the eastward was formed of mountain limestone; that to the westward of 2-inch granite, and that in the centre with blast furnace slag. The work was commenced on the 14th June and completed on the 29th July, 1905, and the portions laid with slag and limestone were carried out without any interruption from wet weather.

That laid with granite, however, was seriously interfered with by rain. It was found that, whereas in the case of the granite the tar did not appear to soak into the stone at all, both the mountain limestone and the slag appeared to be to a certain extent impregnated with the tar.

Within a very short time after it was laid the section of granite showed signs of breaking up, attributable, no doubt, in part to the bad weather experienced when it was being put down, and on the 2nd September last an order was sent to repair it.

But in the early part of October it again gave way, and on the 17th of that month it was in such a state that it would have been taken up and relaid immediately had it not been for the fact that it would not have been possible to get the tar material in time to have the road finished and opened again for the Lord Mayor's Show.

Mr. Jebb. Before 9th November, however, the limestone section gave way also, the side of the road nearest the river going to pieces just as if it were ordinary ballast. As I was indisposed to spend any more money on the experiment, the order of the 17th October was cancelled, and instead instructions were given to take up both the granite and limestone sections and relay them with 2-inch and 1½-inch Guernsey granite to form an ordinary macadam roadway. The central section laid with slag has still some life in it, and I do not propose at present disturbing it.

The result of the experiment would not indicate that a tarmac pavement is suitable for a thoroughfare having the heavy fast traffic of the Victoria Embankment, and I fear even if its durability were greater there would always be a difficulty in paving so large an area (49,000 square yards) in a satisfactory manner, seeing that a continuance of fine weather while carrying out the work appears to be essential for the construction of a satisfactory roadway.

The cost of the experimental tar paving was as follows:—

	£	s.	d.
Paving with aggregate of mountain limestone . . .	221	2	6
" " " blast furnace slag	221	2	6
" " " 2-inch granite	257	13	3
Total	699	18	3

The Works Department's accounts are not exactly complete, but the above figures may be taken as very closely representing the actual cost of the work.

The area of each sample of paving was about 700 square yards: the cost per square yard is therefore as follows:—

	s.	d.
Mountain limestone	6	4
Blast furnace slag	6	4
2-inch granite	7	4

In each case the tar paving was laid 6 inches thick, and the cost of removing the old material is included in the price given.

The actual price charged by the contractors who supplied and laid the tar paving was, in the case of the mountain limestone and slag, 4*s*. 6*d*. per superficial yard: in the case of the granite 5*s*. 3*d*. per superficial yard. The cost of the tarred material delivered on the ground was about 6*d*. less than these figures.

February, 1906.

Mr. Blackwall. Mr. Blackwall, in reply, was much gratified that his modest suggestion for the improvement and saving in cost of maintenance of certain country roads, although meeting with no approval from either the makers or the users of roads, should have borne a part in bringing forward a discussion of such general interest by speakers of the highest authority on the construction and organization of highways. With regard to the points in the Paper selected for criticism, the camber shown in *Fig. 1* had attracted universal scorn, except from two speakers who had rightly surmised that the curvature was exaggerated in the Figure in order to bring prominently to notice the intention that both the heavy and the light road should be convex, and that any idea of a concave road would be unsuitable. To the best of his knowledge, the idea of a concave road had been suggested first by the surveyor to an urban

authority in order to facilitate washing and to add to the comfort of the foot-passengers using the side-walks. These ideas seemed to be eminently urban and not to apply in the case of the distinctly rural roads originally under discussion. A grave objection to a concave road lay in the fact that the traffic naturally gravitated to the centre in order to avoid the cross gradient; this was the lowest part of the road, and therefore the most inclined to be wet and to wear most quickly. The grates would lie in a slight hollow after the road had been made up, and would be inclined to stand up above the macadam as that wore down, and they were exactly in the line which horses naturally followed. It was surprising that some of the varying factors which should govern camber had not been mentioned. It was quite wrong to attempt to lay down a fixed cross fall for all cases, except that from the user's point of view the road should be absolutely flat. From the road-maker's point of view the cross fall had to be taken between the minimum necessary to drain the road and the maximum which would admit of the traffic wearing it equally throughout its breadth. The cross fall must be greater when the material used was soft and wore quickly, so that the camber might be at its best when half-way was reached in the life of the coating. A road which was much sheltered from wind and sun by overhanging trees or rocks, or one which from climatic causes had less chance of drying, required an increased cross fall to throw off the water. On a steep hill the cross fall should be increased, since the surface water would run along the road down-hill in spite of a cross fall which would be enough to throw it off where the road was on the level. The traffic, too, would not keep to the centre up or down a steep hill. The question of camber was not affected by dividing the light from the heavy traffic. The suggestions that a steel-plate track should be provided for heavy traction-engine traffic, and that one of Val de Travers asphalt should be provided for motor-cars, surely pointed to the ultimate advisability of separation. Steel-plate tracks had been tried in Glasgow 50 years ago, and more recently in Germany and New York; while the most complete trial had been made at Valencia. All these cases had been in towns where the hardest sets had been laid adjoining the plate, except at Valencia, where only one class of slow, heavy traffic was provided for, which did not have to turn out for passing other vehicles. On a macadam road steel plates would attract the traffic and accentuate the wear in their immediate vicinity, so that the macadam would be most worn at the edge of the plates, which did not wear at all, producing the unhappy result of a rut with a steel edge. Exactly the same would happen at the edge of a central strip

Mr. Blackwall.

Mr. Blackwall. of Val de Travers asphalt. A steel surface was always bad for horses with iron shoes, and it was hardly probable that horses would universally be shod with india-rubber in the near future. The nearest approach to divided traffic at present to be met with occurred in the Indian roads described by Mr. List, where the light, fast traffic travelled along the metalled centre of the road, and the slow, heavy traffic at the unmetalled margins. This would be satisfactory also in England, but here some system of compulsory separation must be adopted to take the place of the natural preference of bullocks for the softer ground at the side of the road. In England every traction-engine and heavy cart would be found in the centre of the road and motor-cars would have to turn to one side to pass them; while flocks of sheep and cattle would wander in front of motor-cars just as they did under the present system. The wire fence suggested had been objected to by several speakers on the ground that it was not capable of preventing a motor-car from running over foot-passengers on the footpath, or of keeping cattle, and particularly dogs, from straying on to the light road. This was a detail open to any improvement, economy being the ruling idea in the type suggested. A fence capable of stopping a heavy motor-car which ran into it at a high speed would be a very substantial structure indeed; but there was no intention of attempting this. As a check, however, on cattle straying over the light road, a four-strand wire fence would be a very considerable safeguard and advantage, both to the driver of motor-cars and to cattle-drovers, who were still entitled to consideration in the design of country roads for all classes of users. It was very unfair for a motorist to say that people must learn that a country road was no more safe for live stock than was a railway. A railway was intended for no other traffic than railway-trains, while the road was the only way by which cattle could be moved from one place to another. A different view might be taken if a separate track were provided for fast traffic only, and a drover might reasonably be expected to take proper care that his cattle kept to the road intended for them. There seemed to be a tendency in the views expressed by motorists to expect the community at large to provide, all over the country, roads suitable for their use, and equal in width to a broad London thoroughfare. If they cared to make such roads at their own expense for their exclusive use, it would seem reasonable enough that they should have them; but a recent attempt in this direction pointed to the fact that the numerical and financial strength of the motorist community was not yet capable of carrying out such a scheme unaided. Until this was the case they must be content with their fair share of the road

Mr. Blackwall.

with the community at large. Sir John Wolfe Barry missed the point in making the obvious assertion that a good road 42 feet wide was better than two roads 21 feet wide. If a good road 42 feet wide was able to carry both traction-engines and motor-cars without producing mud and dust, and the ratepayers were content to pay for its maintenance, Mr. Blackwall would be the last to suggest placing a footpath and two extra gutters in the centre of it. The road with which it was suggested to deal, it must be borne in mind, was 26 feet wide, always either muddy or dusty, had no footpath, and cost about twice as much to maintain as the twin roads would cost. Mr. Darbishire appreciated the fact that the dust nuisance was caused mainly by slow, heavy traffic pounding up the road-material and the tires of fast motor-cars raising it in clouds; but he did not follow up the idea to the conclusion of the possible advantage to be gained by separating the one class of traffic from the other. It was suggested in the Paper that the road for slow, heavy traffic only could be maintained at a cost of £32 per mile, not, as Mr. Humphreys misquoted, that this sum was the amount spent at present on county main roads. The treatment of bridges would come under a separate heading in the improvements of country roads; in the consideration of a twin road a bridge would be classed for the present along with villages and confined spaces where the separation would be discontinued. He had no intention of doing away with retaining-walls; he merely pointed out in the Paper that, in selecting which side of a given road should be made into the heavy road and which into the light, the heavy road should be laid on the side away from the retaining-wall, so that the heavier weights might be on the more solid ground. If equal amounts of retaining-wall occurred on the two sides, no preference on this ground would be involved. At a crossing of two roads, as suggested by Mr. Shelford, where both roads were of such importance as to have been made into twin roads, it was probable that a speed-limit would be necessary; in any case there need be no more confusion than at a similar corner where two ordinary main roads crossed. At a branch road, or a cross road, or at a gate leading into a field, the footpath would cease in exactly the same way as it would cease if it were next to the fence at the side of an ordinary road, and the foot-passenger would not have to go into the road among the traffic any more than under the present system, except in a case where the footpath had originally been on one side of the road and a branch road on the other; here the central footpath would have to be crossed. At the tops and the bottoms of hills, the only traffic which would cross the footpath would be the light carriages whose drivers preferred to move at a walking pace. The fence would stand a much

Mr. Blackwall. greater chance of being knocked away without anyone knowing anything about it if the footpath were placed at the side of the road than if the fence were placed on a raised footpath in the middle of the road. Foot-passengers only wanted to be at the side of a road where there was something at the side to attract them, as for instance in a village, in which case the twin road would be discontinued. People travelling on foot along a country road were, as a rule, not numerous, or else not particular; and it must be remembered that on many country roads there was no footpath at all. There seemed to be in the minds of several of the speakers an idea of town conditions on country roads, and traffic was spoken of as though it were made up of the crowded procession of a busy London thoroughfare. With the gradual increase of future traffic, such country roads might well be handed over to the care of a central authority, and maintained at a high cost with expensive materials for a width of 42 feet. But there were at present a vast number of quiet country roads, such as he had had in mind when writing, in the maintenance of which economy was an important object, on which there were very few foot-passengers but heavy carts and traction-engines enough to cut up the surface, and motor-cars enough to cause a nuisance by raising the dust—enough to be entitled to a smoother road-surface than the traction-engines left, and enough to require a free course unobstructed by cattle. Under such conditions, and with circumstances favourable for its construction, he still ventured to think that the twin road might be a source of comfort and economy.

Correspondence.

Mr. Aitken. Mr. T. Aitken had found that spraying tar on macadam roads in connection with re-coating and rolling was very successful. Tar and chips and dust (from stone-breakers) took the place of water and the ordinary binding materials (sand, loamy sand, road-drift, etc.) and roads so treated showed neither dust nor mud. Although the initial cost was slightly higher, the life of a road would probably be at least doubled.

Mr. Bennett. Mr. F. J. Bennett, late of H.M. Geological Survey, considered that, had the principles laid down by the great road-engineers, Telford and Macadam, been followed by their successors, and had there then been a Government department charged with the

construction and maintenance of roads, there would have been a proper system and valuable data to help at the present juncture. The neglect of the roads was largely due to the advent of the railways, though it ought to have been foreseen that increased railway-traffic must be followed by increased road-traffic. Many years' consideration of the best shape for a road had convinced him, both as an old cyclist and as a practical geologist, that the universal crown system for roads of all widths and gradients was a mistake. He had long considered that narrow roads in fairly hilly districts would be much better if made with a uniform transverse slope and without ditches, the slope of the roads coinciding with the general slope as far as possible. Such roads would be wider and stronger, and the traffic would not be forced to the centre, making thus one track for the horses and one for the wheels, with all the difference in wear that this entailed. The wear would be more even, and the water would have a broader distribution, so that its velocity and erosive power would be much diminished: the tendency would be for the water to wash off the mud and so diminish its accumulation, thus reducing the cost of cleaning and hastening the drying. Having done a good deal of geological work in chalk districts, he had there found that considerable lengths of roads running along valleys were on the cant, and had then discovered that these portions coincided with and occupied the old cultivation-terraces or linchets. These linchets always sloped with the hillside, and thus roads based on these terraces were similarly inclined. Dust had a manurial value, and he had long advocated that it should be swept off, stored in bins on the adjacent land till wanted, and then applied to the land. The removal of mud from first to last was a complicated and most expensive matter. Better road-material, and waterproofing it, which would often improve the inferior kinds, would of course reduce both dust and mud. Formerly road-material varied much more with the district and the authority than it did now, the nearest available metal being used—as it still was in places—without any regard to its wearing properties; so that the utmost diversity of road-surface was the result, and some portions of road cost much more than others. But more uniform metal was now the rule, and marked improvement in the roads was the consequence, as, for instance, in parts of Norfolk, where roads which were once very bad were now very good, thanks also to the use of the steam-roller. To decide upon the best material for use was a difficult problem. What was best for motor-traffic was often too hard for horses' feet, and it might prove necessary to set aside special tracks for each class. Study of the geology of the road had been prac- Mr. Bennett.

Mr. Bennett. tically ignored, with much resulting loss of money and of efficiency. Proper attention to this would often furnish the clue to unequal wear, and the reason why one part of a road cost more than another. Now that most of England had been surveyed geologically for drift, all road-surveyors' maps should be geologically coloured, to show the subsoil; and the metalling of the road should be in accordance with the subsoil, and not be put down regardless of it, as was usual. He was also inclined to think that the steam-roller was not the ideal consolidator, and that a steam-rammer might give better results, the ramming to be adjusted to the thickness, etc., of the metal. Indeed, all repair patches should be hand-rammed. If ramming preceded rolling, better consolidation, and thus a better surface, would result; also, lighter rollers could then be used, with less risk of damage to water-pipes, etc., and less crushing of the metal, which entailed waste and increased the formation of dust and mud. There should be some means of adjusting the weight of the roller, especially where the road-bottom was soft, as a heavy roller often left a wavy surface behind where the bottom was soft, much to the detriment of the road. A geological road-map would often show where these soft bottoms might be expected. "Extraordinary traffic" often meant weakness of the road; its causes should be looked for and the road strengthened. In the Roman military roads there was little chance of this occurring, as the reserve strength of these roads was immense; there was much to be learnt from them. The existing roads, being of course for business purposes, mostly followed valleys or went directly across country. Yet how many lovely spots there were where strictly pleasure roads might be made—preferably turfed—and kept firm and good by rolling. Such roads as these might easily be made along rolling turf-downs, and along the edges of escarpments overlooking the vales beneath. Many people, he thought, would gladly pay a moderate toll to be able to ride along such high-level roads, on the firm springy turf—truly fragrant, dustless and mudless roads.

Mr. Brackenbury. Mr. C. E. Brackenbury remarked that it almost seemed as if the matter for immediate consideration was not so much the question of the construction and management of existing country roads as the question how horse-drawn vehicles could be most quickly replaced by motor-vehicles, and what kind of roads would be most suitable and economical for the mechanically propelled vehicles of the near future. For, so long as there was any considerable use of both these systems of locomotion, it seemed to him it would be impracticable to have the most efficient kind of road. The highway must be made to suit the traffic, and until motor-traction largely predominated, the ideal road for it would be difficult to secure.

Mr. Brackenbury.

The subject appealed to him more particularly as a gas-engineer. The advent of the heavy motor-wagon or lorry had meant many breakages of gas-mains, often without the slightest means of discovering the offender, and perhaps without even the discovery of the rupture of the pipes until long after the occurrence. The gross weight of the motor-wagon or traction-engine was considerable; according to his own experience in buying a steam-wagon for coal and coke, for a net load of 3 tons, the gross weight was between 5 and 6 tons; while a trailer behind for carrying an extra 2 tons brought up the total gross load to about 10 tons. The heavy part of the twin road suggested by Mr. Blackwall would at least obviate such breakages, though Mr. Brackenbury considered the twin-road idea to be altogether unpractical and unadvisable. A second point which particularly interested the gas-engineer was the question of using tar in some form for waterproofing and dust-preventing. Having lived and worked in France and Italy, he had taken special notice of what had been done in this matter in those countries. Various experiments with tar-washing of roads had been carried out during the last 4 or 5 years. The results showed generally that the durability of roads was undoubtedly increased, the interval between repairs being prolonged from 3 years to 5 or 6 years, so that there was every prospect of the total outlay not being increased. The cost of thorough tar-coating varied: it was 7 centimes per square metre at Meaux, 12 centimes at Melun, and about 14 centimes at Versailles (respectively 0·55*d.*, 0·96*d.* and 1·11*d.* per square yard). The quantity of tar used per metre of road-surface was 1 to 1½ kilogramme (1·84 to 2·76 lbs. per square yard). Similar trials had been made also at Rome, Bologna and elsewhere. In England the use of tar or "tarmac" seemed to meet with general approval, which the gas-engineer gladly welcomed, for it promised to give him another outlet for one of his by-products. From a legal point of view there was undoubted need for consolidation and revision of the Acts relating to roads, for there had been no general Act since the General Highway Act of 1835, though there had of course been many amending Acts and Orders. Mr. Jebb's instance of a road maintained by half a dozen different authorities was interesting, though perhaps matters were hardly as bad as they looked at first sight; for probably some of these authorities, as was customary, arranged for others of them to carry out the works of repair, in which case there would not be six different authorities doing the necessary work. At the same time, he supported Mr. Jebb's contention that the county councils should swallow up these smaller authorities, so that the work might be done more cheaply and uniformly. But he would go further than this and suggest

Mr. Brackenbury. governmental control of the main roads, on the lines of the *routes nationales* of France, Italy, and other continental countries. The matter was one of national importance, and civil engineers could in this way aid their military confrères in facilitating war-transport and coast-defence.

Mr. Dryland. Mr. A. DRYLAND considered that if the whole of the roads in a county were placed under one authority they would not only be maintained in a more uniform and satisfactory condition, but also at less cost. The real hindrance to any great improvement of country roads was a monetary one. So long as practically the whole of the cost in rural districts had to be provided by rates upon the land, it could not reasonably be expected that the already over-burdened and unprosperous agricultural community would rate itself to provide roads of excellent surface in compliance with the demands of motorists; especially as the present form of road answered all the reasonable requirements of the agriculturist. Either a considerable tax must be levied upon motors and vehicles generally, which should be devoted to purposes of road-maintenance and improvement only, or a large portion of the cost must be borne by the central authority and be met by taxation, as distinct from rates: in the latter case it would no doubt be necessary to make payment of the cost depend partly upon the attainment of a standard of efficient maintenance. More labour could no doubt be usefully employed in sweeping roads to keep them free from dirt and dust. On the other hand, with the roads as at present constructed, too much sweeping would be detrimental, by removing all the smaller binding material, and in dry weather the surface would disintegrate. There was little doubt that tar-macadam would be perfected and be largely used, in which case the dust and dirt would be minimized. The additional first cost of this change of method could probably be met by degrees in urban districts, but not in the purely rural districts until the cost was put upon shoulders better able to bear it. He did not believe that Mr. Blackwall's proposal for a double road for country traffic was feasible, on account of its cost, which he thought had been much under-estimated. The assumption that the landowners would give the necessary land was unlikely to be realized; nor did Mr. Dryland see why they should be expected to contribute in that way for the benefit of the general public. Motor-traffic, which really was the cause of the demand for improved roads, was not of a local nature; and what a public-spirited landowner might be inclined to do for the benefit of his own neighbourhood he was not likely to do for the advantage of motorists from all over the country. Mr. Blackwall's estimate

Mr. Dryland. could in any event apply only where the road was practically level with the adjoining land; but this was the case on but a very small portion of the roads of the country, most of them being considerably below the level of the adjoining land; and therefore widening would involve a large amount of excavation and haulage. Where the roads were above the level of the land, expensive retaining-walls and fences would be necessary, none of which were provided for in the estimate. Even Mr. Blackwall's £2,000 per mile, as the initial outlay, could be borne only by very few localities. Mr. Blackwall's figures for the present cost of maintaining main roads were wide of the mark, the average cost of rural main roads in Herefordshire being £40 per mile, and this represented the cost of roads carrying comparatively light traffic only. In some counties the cost was three or even four times as much as this, the average for the whole of England being, Mr. Dryland believed, more than £60 per mile. The suggestion to allow the heavy traffic to consolidate the material was neither economical nor desirable; in fact it would not be tolerated at the present day, when steam-rolling had become essential.

Mr. Hawkshaw. Mr. J. C. Hawkshaw, Past-President, had had some experience of country roads, as his home of late years had been almost wholly in the country, where, moreover, he had engaged in farming on a somewhat large scale. Agriculture was an industry which was affected not only by the condition of the roads but also by the cost of maintaining them. He had long been of opinion that the management of the roads should be more centralized than it was. He had a good example of divided jurisdiction near home, as the mile of road between his house and the railway-station was under three authorities—two county authorities and the railway-company, which repaired the road where it crossed the railway. The two county authorities repaired the road with local metalling, one with good material, the other with bad, so that ½ mile of the road was always in good order and rarely required attending to, while the other ½ mile was always having money wasted upon it and was more or less out of repair. The metalling used in the latter case was a soft sandstone, much of which was reduced to sand by the men who broke it, and most of the remainder was crushed to sand by the first wheels that passed over it. He feared that local authorities were, as Mr. Jebb said, often tempted to use local materials even when not the best for the purpose in view. A more central authority would take wider views. The question of widening the roads was not such a simple one as might appear from the section (*Fig. 1*) given in Mr. Blackwall's Paper. Such a section would not be possible except at a prohibitive cost in many parts of the country; certainly not in Sussex and in many

Mr. Hawkshaw. other parts of the country with which Mr. Hawkshaw was acquainted. Many of the Sussex roads were in cuttings 15 or even 20 feet deep, worn down by the traffic before the days of metalling. The process of deepening was still going on in some of them where no fresh metalling was put on, the ruts in the sand alone being filled from time to time. Again, it must not be assumed that landlords were likely to find land free of cost to further the making of double roads with a wire fence between them, along which motors might travel at high speed. Much motor-traffic already tended to depreciate property along some roads. He had spent a good deal of late years in improving public roads in his neighbourhood, and a thankless task he found it; but he would do nothing to encourage high-speed traffic on the roads. He quite appreciated the advent of motors, their great convenience and usefulness; but it was no good overlooking the fact that motor-traffic in its present stage had already done much to destroy the pleasures and comforts of country life. If high speeds were to be maintained it would be necessary to provide special roads to accommodate them, as had been done for railway-traffic. He had known motor-cars travelling on the Portsmouth road to beat the trains on the London and South-Western Railway between points two or three stations apart. As the number of motors increased, such roads would become impossible for other traffic. It must not be forgotten that England's greatest industry was agriculture. It was to serve agriculture that most of the roads had been made, and it was for agriculture that most of them must be maintained.

Mr. Heslop. Mr. T. H. B. Heslop remarked that, but for the advent of the high-speed motor-car, it was possible that present conditions might require little alteration beyond the gradual strengthening of existing roads. Traction-engine traffic had been general for many years and had been fairly well provided for. In cases where this had been considered "extraordinary" he had generally found that compensation for the resultant damage had been paid to the highway-authority. The heavy motors which were now coming into general use were likely to cause damage owing to their greater speed and weight, although they were subject to considerable restrictions as to loads and width of tires. This class of traffic, however, was local in character, and the owners contributed to the cost of local road-maintenance; moreover, it was to be found in excess only in very few districts of England. It was therefore the high-speed motor which required more immediate consideration. The use of such motors would increase rapidly, and it was quite evident that even the main roads had not been constructed to carry traffic

of this kind with safety and comfort. With few exceptions, these roads were too narrow, winding, and badly graded, having probably been originally formed merely by custom and for local convenience, without regard to through traffic. As a rule they were made with local materials and were without properly constructed foundations. The following brief scheme was suggested as containing a basis for improvement in present conditions:— Mr. Heslop.

(1) The metalled portion of the important main arteries or through routes to be widened and improved. This could frequently be done at a comparatively small cost, as roads of this class had generally grass margins of some width.

(2) The best material for maintenance, namely, selected granite, to be brought into general use and to be applied with the least possible quantity of fine chippings or grit when steam-rolled. Under these conditions a very small amount of dust was formed. He had recently tried a mixture of tar and pitch brushed into the interstices of the metal after well rolling, and so far it had proved satisfactory, there being little or no dust visible.

(3) Through routes to be maintained directly by Government, through a properly constituted Roads and Bridges Department. This would secure the much-desired uniformity of treatment, and the cost would be borne by the country generally, which seemed reasonable, seeing that the traffic which caused the altered conditions was no longer local.

(4) The remaining roads to be placed under the control of one authority only in each county, in order to secure uniformity and economy of management. Possibly it might be an advantage if one-half of the actual cost of the more important of these main roads was repaid by Government, on condition that they were maintained in satisfactory order.

Considerable difficulty was experienced in obtaining adequate funds for ordinary road-maintenance, more especially in purely agricultural counties; therefore it was not to be expected that they could raise the large sums now required for purposes of improvement and strengthening. He found that Mr. Blackwall had considerably under-estimated the cost of his proposed twin roads: the material and labour would cost more than he allowed; and Mr. Heslop much questioned whether land would be given for the purpose of widening. In any case, he was afraid the scheme was impracticable as a general one throughout the country if the money had to be provided by local authorities; and if it were not general it would of course be of no advantage. It had been suggested that tar-macadam might be

Mr. Heslop. used in order to obviate the dust nuisance. Several kinds of this material had come under his observation, but he had not yet seen a really successful example under all conditions of traffic and weather. For general use upon country roads the cost would be prohibitive.

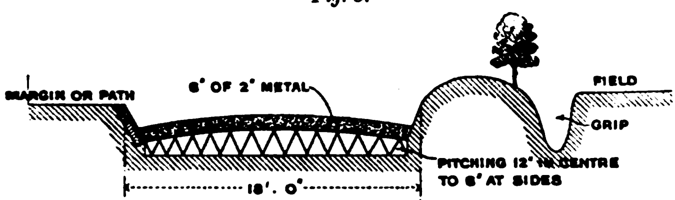
Messrs. A. E. & P. H. Hurse. Messrs. A. E. and P. H. HURSE agreed with Mr. Jebb as to the desirability of placing all country roads under one central authority in each county. With his remarks on scraping, however, they did not agree, as it was found in the country around Dublin to be absolutely indispensable under certain conditions, for example, early in the winter when the summer crust or skin lifted with the moisture, and after frost in the early spring when there had been a light shower. It was necessary to be careful not to tear the surface, but if the mud was not removed when it was in this stiff state and was allowed to wear back into the surface again, it would do immense harm; and should the weather revert to frost after the thaw, the whole surface would be lifted on account of the thick coat of mud. Too much sweeping was bad in the summer, as it was apt to leave the surface bare and to tear the grout from between the stones. Although there was a similar example of divided authority on the Scalp Road, on the boundary of the counties of Dublin and Wicklow, the writers believed that instances such as that mentioned by Mr. Jebb were rare.

Mr. Blackwall's suggestion did not strike them as being practical, as a system of twin roads would only tend to increase present difficulties. Among the objections to it were the following facts: for miles before the centres of population were reached, the sides of the roads were lined with houses, frequent even if not continuous, and generally of a high-class character; where the main roads could be adapted to this system there was no need of it, as traffic was scanty and houses scarce; property-owners were not likely to give the land, especially upon the motor-car roadside, as the value of the property would scarcely tend to increase owing to greater dangers and irritating restrictions; the ratepayers would naturally resist any expenditure upon the special formation and exclusive use of roads for motor-cars and quick traffic; as steam-rolling had been introduced and adopted upon proof of the extra loads that could be borne upon a good road, farmers and other users of heavy traffic would expect the heavy road to be rolled too; and lastly, the central path would be inconvenient and very unsuitable in wet weather. They were of opinion that the only method was gradual improvement and widening of existing roads, as suggested by Mr. Jebb, and, should the dust-preventives be proved to be satisfactory, economical, and lasting, their use upon roads within a

Messrs. A. E. & P. H. Hurse.

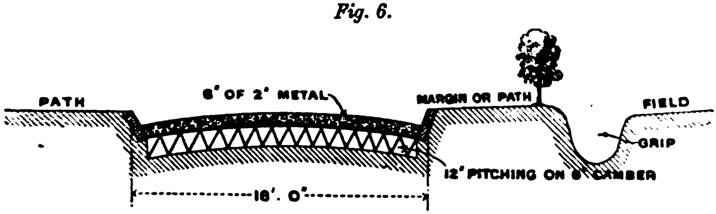
certain distance of each town and village. In Ireland country roads were all under the county councils, who also maintained the road-approaches to bridges over and under canals and railways, the expense being levied in such cases upon the owner *pro rata* according to the cost per perch of the adjoining road. The maintenance of the roads was put up to tender, the county surveyor having the power to submit an estimate of the amount for which he would carry out the work by direct labour; and it was found that roads having heavy traffic were frequently maintained

Fig. 5.



by this method. Roads were not classified, as the dual system of control did not exist. While it was not possible to compare the cost of direct labour versus contractors, as conditions varied upon different roads and even upon the same road from year to year, it might be stated that the roads in a certain barony which cost £6,000 in 1898 under contract were estimated to cost £5,040 in 1906 under direct labour. Both systems were open to abuse, and proper and adequate supervision was required on road-maintenance as in any

Fig. 6.



other branch of engineering. Even the labour should be skilled. Steam-rolling was in vogue in certain counties, and had tended to produce a much more efficient and lasting road than heretofore. There was little experience of dust-preventives in Ireland; one instance, that of the use of "westrumite" on the Phœnix Park roads during the King's last visit, and upon the Gordon-Bennett course, was not likely to be repeated on account of cost and want of durability. There was very little road-forming, but the accompanying sketches (*Figs. 5* and *6*) might be taken as typical of Irish

Messrs. A. E. & P. H. Hurse. practice. The camber was generally 6 inches in the 18-foot road, from crown to channel, and had not been found to be excessive by motorists. The plan illustrated in *Fig. 6* afforded the better drainage. The system of road-maintenance varied in different counties, and even in the same county one overseer would give his work more rolling and less spoil, another would use more water, and so on. Patching was in vogue in parts of the county of Dublin, but not in the better-class districts except on mountain roads. Road-spoil was undoubtedly the best for blinding and binding the metal, sand having no binding-qualities. Drainage was half the battle in road-maintenance: a road which could not be drained would never be worthy of the name. Most country roads were devoid of foundation, and it was out of the question, on account of expense, to reform them; therefore the best means of drainage available must be adopted. All roads must be convex in transverse section, and the water-tables should be so formed as to carry the water quickly to the nearest hollow and thence to the ditch; this done, a fair surface and good road might be looked for, with reasonable care. One great mistake was that after the road had been finished off with the roller, it was almost invariably left to take care of itself, and no other reason could be advanced for half the bad rolled roads. If a little attention were paid at the right time, the life of most roads would be doubled, if not trebled. In wet weather the road should be swept, and any depression found in the surface stopped at once with screenings or 1-inch pebble gravel. If this were not done the water would percolate through the new coat, and then the crust became broken and the road wore out before its due time. In the county of Dublin nearly all roads were steam-rolled, the whole surface being first scarified with a scarifier attached to the roller. The cost of this was $\frac{1}{8}d.$ to $\frac{1}{4}d.$ per square yard, against $\frac{3}{4}d.$ if done with the hand-pick. The largest of the old stones were raked to the centre of the road, the new coat of stone, broken to pass a 2-inch or $2\frac{1}{2}$-inch ring, being spread. It was very important that all macadam should be turned in the spreading, because if it were only levelled with the back of the shovel, all the smallest of the load would be found to be in one place, and after rolling there would be a depression at this place. The new coat was then dry-rolled until it was well consolidated, after which the road-scrapings, or spoil for blinding, having meanwhile been worked into a thin putty or sludge in the water-table or channel, was swept to the road centre from both sides. By thus rolling, the stones were keyed together and were not liable to loosen or ravel up afterwards. Care must be taken not to use too much water, and this was doubly necessary when applying a

Messrs. A. E. & P. H. Hurse.

heavy coat, as it could not be properly consolidated, and the road had to be left until the water drained off. The road was gone over again after 4 or 5 days, and by washing off and re-rolling, the new coat was the better consolidated. In the case of re-rolling a road that had worn badly, scarifying was generally indispensable, and saved much macadam; but when rolling in a heavy coat on a soft foundation it was not necessary. The life of a road naturally depended on the nature of the traffic. The Barony of Rathdown was largely residential, and the vehicular traffic was mostly spring-borne and essentially light; consequently the life was longer than in districts where there was heavy cart-traffic, as in North Dublin. A main road in South Dublin kept in first-class condition lasted about 3 to 5 years, and by-roads lasted 5 to 10 years. The Templeogue road, however, required to be rolled yearly. Irish roads were not generally noted as good, but South Dublin roads were an exception, universal praise being awarded to them by motorists. There were nearly 90 miles of roads in one barony, and of these about 18 miles were rolled each year. Cost depended upon the prices of materials and upon the method of rolling. In one case in South Dublin it cost £12 to roll 300 lineal yards of road 18 feet wide, and as it took about 1 cubic yard of 2-inch limestone macadam at 4*s*. 6*d*. to cover 1½ lineal yards, the total cost for 300 lineal yards came to £57, inclusive of labour, etc., or roughly £335 per statute mile. Motor-traffic was most injurious to roads in the summer months, as the high speed at which these cars were driven caused the tires to tear the surface as well as clear the dust into the channels, leaving the centre absolutely raw; and if not watered the surface was liable to ravel.

Mr. Hutchins.

Mr. G. A. Hutchins remarked that the plan of sweeping the dust in summer from country roads had been adopted with advantage for some years in Montgomeryshire, where and when practicable; but there were two grave objections to its extensive use. In the first place, the expense would unfortunately be largely increased, although no doubt the public would suffer less annoyance. Even if only one more man per mile were required for a quarter of the year, it would mean an additional cost of more than £4,000 per annum on the 400 odd miles in this county, or more than a 4*d*. rate. Secondly, excessive sweeping loosened the top crust of the metalling. No mixture of stones of different quality answered in practice, as the surface wore into holes. In hilly districts the improvement and modification of gradients of existing main roads, by short deviations, was certainly the first essential. Under the old South Wales Turnpikes Act, a Government inspector of roads used periodically to travel over and inspect the turnpike roads,

Mr. Hutchins. and check the accounts of the road-surveyors; and to Mr. Hutchins's knowledge this system had worked extremely well in improving the class of work done on the roads inspected. As a county road-surveyor, he thought that, in consideration of the largely increased weight of traffic brought, and to be brought, on the country roads by the advent of the heavy motor-car—not to speak of the exigencies of high-speed motor-traffic the larger portion of which was what might be called cosmopolitan, and not local—the Government ought, from imperial taxes, to a large extent (say one-half) to subsidize directly the maintenance of main roads, and, as in the case of the county police, to pay one-half of the salaries of the county and district highway-surveyors. There would then be the right, as under the old South Wales Act, to supervise the work and the surveyors. The supervision of the maintenance of the district and other country highways, and of their surveyors, might, as Mr. Jebb suggested, devolve upon the existing county surveyors, in consideration of a similar subsidy being given from imperial and from county funds. But it would probably be found a more economical, and generally a more workable plan, if all the district councils were still to maintain their own highways under such subsidy and supervision. If a system of Government subsidies were adopted, there would certainly be room for the plea that the general body of taxpayers should have some call upon the work paid for, in respect to the question of the unemployed. But here it would be right to put in a plea also for the poor country ratepayer, who got comparatively little benefit from, and had to pay for, the greater part of the damage to country roads caused by both the heavy motor and the high-speed vehicle. The small farmer must keep his horses still, to do his farm-work, and now he was losing the greater part of the outside draught formerly done on the roads, when not at work on the farm, in favour of the motor: while, at the same time, he had his rates almost doubled to repair and improve the roads, so that they might be able to carry his competitors' extra weight. To give a practical illustration, the 400 miles of main roads in Montgomeryshire, a few years ago, cost £8,000 per annum. In 1906 they would cost, owing chiefly to heavy motor-traffic, £13,000; and for next year, the estimate was still more. This might fairly be urged as a plea for a substantial Government subsidy for the relief of the country ratepayer, as well as for better country roads.

Mr. Jenkin. Mr. CHAS. J. JENKIN observed that the economical repair and maintenance of roads was largely a matter of administration and management. Many roads, especially in country districts, had been allowed to get into such a state of bad repair that the initial cost of

putting them in proper order was almost greater than could be borne by the authority in charge of them; and until such drastic repairs were put in hand, they would continue to be costly to maintain, and muddy, and consequently dusty and generally ill-conditioned. In his own experience the roads in a certain district, which had been starved and were generally in bad condition, were taken in hand and a large expenditure was incurred for repairs, with the result that a great improvement in their condition, and a large annual saving per mile on the previous cost of maintenance and cleansing, had been made. While a very slight camber was sufficient for the hilly portions of macadam roads, on flat portions and easy gradients the cross fall must be sufficient to keep the roads dry. Excessively slight camber was, in his opinion, unsuitable for macadam carrying much or heavy traffic, and roads with such camber should be constructed with a smooth surface and of practically waterproof material, such as wood blocks, tar-macadam, etc. The use of such materials would necessitate a considerably stronger foundation than at present existed on the majority of main or other roads, and the cost would be considerable. The flat road was being asked for by practically only one class of traffic, and it would appear reasonable to expect that automobilists should contribute largely to the cost of altered road-construction by means of increased taxation, payable either to imperial, county or local funds; imperial funds being applied through the medium of a specially-constituted national road-authority. Mr. Jenkin.

Mr. Crawford Lindesay, on returning to England after 15 years spent on municipal and railway work in South Africa, had been struck by two signs of neglect on some country roads; namely, the absence of footways, and inefficient drainage. Owing to the recent development of high-speed motor-cars, county councils should regard a main road in the same light as they would a railway; and it was unfair to the motor-car driver, as well as to the pedestrian, to allow any important road to be without a footway. In country districts foot-passengers observed no rule of the road, and it was surprising that so few people met with accidents. The cost of constructing a footway along the side wastes of country roads would be trifling in most cases. The excavation and wheeling could be performed by unskilled labour, say, by the unemployed, and the top dressing and turf edging could be laid by the regular road-men. Mr. Blackwall would, he felt sure, reconsider the position of his footway, and place it at the side of one or other of his roads, as its central position was undesirable. The drainage of a main road carrying heavy and high-speed traffic must receive nearly as much consideration as that of a railway, if the cross Mr. Lindesay.

Mr. Lindesay. section was to be maintained at a minimum cost. There were important roads within 25 miles of the metropolis which had no footpath, and on which the drainage was practically useless. In some instances he had seen these roads in cutting in a clay district without even a make-believe ditch cut. This should not be tolerated. He did not attach blame to county surveyors for such faults; they were not granted votes for more than the barest necessities; but he thought the Government should assist in the maintenance of main roads. County surveyors should be supplied with more professional assistance, so that the cutting of even a ditch to the nearest clear outlet might not be left to the eye of the road-man only; even a ditch in flat country should be provided with sufficient fall, which must be ascertained by some one capable of using the dumpy level, and between such level-pegs as the surveyor gave, the work should be completed by graded intermediate pegs determined by the use of boning-rods, of which every road-man should have a set at hand. When standing water rose within, say, 2 or 3 feet of the surface, subsoil-drainage was very desirable; not a few shallow trenches along the wastes, but a good, deep ditch or something better. Indeed, he hoped the time would soon come when the inverts of these ditches and part of the sides would be concreted and rendered and floated in cement mortar to preserve the gradients, with perhaps a catch-pit at intervals to arrest debris. Rough concrete inverts did not require much skill to lay, and the cost would be trifling; they could be placed alongside the hedge with a footway on the other side and with sufficient pipe-drains from the curb line. In Natal and in Southern Rhodesia he had had to provide for storm-water drainage on the basis of 1½ inch to 2 inches of rainfall in ½ hour, and under such circumstances concrete channels must precede, or be laid at the same time as, the macadam roadway. Although most people who had not had experience in road-making and maintenance thought any intelligent man would be good enough to employ as a road-man, those who had to supervise roads knew that such a man must possess a good eye and have acute powers of observation. No amount of training would make some otherwise intelligent men fit to have charge of a length of road. Where it was impossible to discharge drainage into a natural watercourse, or where much expense and annoyance to adjoining owners would be entailed by cutting a ditch across their fields or discharging storm-water into their low-lying fields, would it not be advisable to resort to covered soak-wells, where the geological features of the district permitted of such a method of absorption? For instance, in districts near London might be found 6 feet of clay overlying the chalk. In such cases it appeared to him that a steened well

might be sunk to the chalk, and as the upper chalk was often excessively hard and free from fissures, a shallow boring might be carried into the chalk, and a few dynamite cartridges fired at the bottom of the boring. The top of such a well might terminate at the surface with a gully, or a catch-pit might be built near the drain-discharge to prevent the admission of silt or debris. With regard to the cross fall which should be given to a road-section, he thought $\frac{1}{2}$ inch per foot was not sufficient for macadam. For roads of 20 feet or less he preferred $\frac{5}{8}$ inch per foot, and for wider roads $\frac{1}{2}$ inch to $\frac{3}{8}$ inch; the wider the road the less should be the cross fall. He had often observed that re-metalling was performed upon a road-surface which had not been properly freed from detritus; such action was quite inexcusable. County councils should own their own rollers and not roll by contract; under the latter system the roller often left the site when its work was but half finished. In conclusion, he would like to recapitulate that the secret of a road's life lay in the efficiency of its underdrainage, the quality of the metalling, the reasonable fall given to its cross section, the cutting of its hedges, etc., and last but not least, in the ability of those who supervised and those who carried out the engineers' orders. The road-man should be encouraged to take pride in his length as a ganger did on his length of railway. If he found that the life of his road was less in one spot than in another, let him endeavour to discover the cause and report the matter to his surveyor. Mr. Lindsay.

Mr. G. T. LYNAM pointed out that the essentials of good road-construction had been well understood for fully a century. What the road-engineer required was not more theory or more science, but more money with which to put into practice the knowledge he already possessed. The essentials referred to were: (1) perfect drainage; (2) solid foundations; (3) compact and hard wearing surface. Good drainage was generally obtained, but a solid foundation was less frequently met with. Many main roads were completely devoid of foundation other than the natural subsoil, which was very variable in stability, and the thin coat of surface material (granite, limestone, gravel, slag, or whatever it might be) was inadequate to withstand anything but the most ordinary country traffic. Hence the frequent claims of road-authorities for damage by "extraordinary traffic," many of which were nothing less than an admission of the total inadequacy of the roads. Upon roads with unstable foundations a compact surface could not be maintained, and the attrition of the stones forming the surface one against another was the main cause of dust. Another frequent cause was the improper application of the metal when the road was repaired. Mr. Lynam.

Mr. Lynam. Metal should not be laid down more than 3 inches thick at one time, and it should be thoroughly rolled and compacted before another coat was added. As small a quantity of binding as practicable should be used, and it should be of the same nature as the coating itself. Tar-macadam was no new material for surfacing roads; it had been used extensively for upwards of 30 years. If it had failed anywhere it had been from disregard of one or more of the three essential principles of good road-construction mentioned above, or of the following requirements. The metal should be of such a character that it would readily assimilate the tar compound on its surface; the tar compound should be crude tar boiled and thickened with pitch (not pitch and oil); and the finished surface of the road should be made without the addition of a coat of tar asphalt, but with only just sufficient tarred chippings (of say ½-inch gauge) to fill all interstices. A road so made would be impervious to rain; being always dry throughout its entire mass, it would not break up after frost; it would be practically free from dust; it would require little or no sweeping and no watering; and, if made of proper materials, it would not become greasy unless mud was drawn on to it from the surroundings. The best material at present available was selected hard furnace-slag, which had the property of remaining gritty under wear. One road bearing chiefly light traffic which was laid with this material in September, 1901, had not since cost one penny in repairs, except where it had been cut up for drain or other services. The original cost of the surfacing only (4 inches thick) was 2*s.* 3*d.* per square yard, or, say, £1,400 per mile for a 7-yard road.[1] The chief objection to tar-macadam roads was that they readily became slippery in hoar-frosts, and another drawback was that they were not suitable for ordinary traffic upon gradients steeper than about 1 in 30.

Mr. Marten. Mr. E. B. MARTEN, whose work upon roads had been largely connected with intruding under them with large water-mains or sewers, which however carefully done must cause mischief, thought it would be well to give some consideration to the question of the best way of constructing the roads so as to provide for meeting the difficulty by laying the central path of the suggested twin road with special material easily removed and made good. The subject seemed to possess so wide an interest that its consideration might well be renewed when the present Papers and the discussion upon them were in the hands of all the members.

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[1] A piece of this surfacing, forwarded by Mr. Lynam, was exhibited when the Papers were under discussion.—SEC. INST. C.E.

Mr. JAMES PERRY observed that he regularly had charge, in the west of Ireland, of more than 1,200 miles of roads maintained by a county council. Many of them, there maintained as public roads, would be accommodation or private roads in England. The average cost of maintaining these roads, including main roads, was between £8 and £9 per mile per annum. They included more than 100 miles of good motoring roads which were maintained for £8 per mile per annum. The ordinary traffic was light, the local stone was of good character, and it was easily obtained in a suitable form for mending roads. Granite was found in some parts of the district, and in other parts there was a quartzose schist, breaking easily into rhomboidal fragments, which was, he thought, the best stone for a macadam road. When it pulverized, this stone did not reduce to mud, but became fine sand and allowed water to pass freely through it; so that almost as soon as rain ceased, the road was dry, or at least comfortable to cycle on. A large part of the area was carboniferous limestone, and this material was in parts excellent for roads of light traffic, as most of these roads were, but some of the stone was not good. To bring good material to places where the local material was not good, as suggested by Mr. Jebb, would be impracticable, owing to the expense. It could not be laid down like a rule of right or wrong conduct how a road should be maintained; the expense of maintenance must bear some reasonable relation to the road's utility, and to the amount of traffic upon it; and in many cases it was necessary to be satisfied with less than perfection. Roads which cost £40 per mile per annum and upwards deserved careful attention; the fresh applications of macadam should be rolled in, and the surface should be kept brushed clean; and it would be found that, apart altogether from the saving to the public in the wear of carts and carriages, the maintenance of the road would cost less. One great source of expense in maintaining roads was the cost of supervision of labour. The employment of small gangs or isolated men was not economical; and the variation of the weather rendered it difficult to arrange constant employment for the staff. It was easy to say how macadam roads should be formed and maintained; the real difficulty was to manage their maintenance economically. Certainly they would never be maintained so that no person would ever complain of occasional dust or dirt. He had no faith whatever in any thin-layer applications to road-surfaces to prevent dust; but he had no doubt that a good heavy coating of tar-macadam would be effective, and this was a question of cost. In some cases road-rolling was not done scientifically. Where the same macadam Mr. Perry.

Mr. Perry. was used over and over again—loosened by scarifiers, rounded with repeated disturbance, and filled with mud laid in too thick a layer—the roller pushed it into waves transverse to the line of road; the mass itself was bad, and the waves, barely noticeable at first, became very bad when worn by traffic. He had noticed work of this kind in London. The layer ought not to be so thick as to permit this jamming-forward effect of the roller, the fragments should be angular, so that they would interlock, and the mass underneath should be as free from blinding as possible; that was, the blinding should be confined as much as possible to the surface. He thought the best short Paper on road-maintenance he had seen was one recently read before the Institution of Civil Engineers of Ireland by Mr. P. C. Cowan, Chief Engineering Inspector to the Irish Local Government Board.

Mr. Blackwall's Paper dealt with a question which would become more pressing as time went on; probably the point of pressure which would result in immediate action had not yet been reached; certainly the stage for a general treatment had not been arrived at. Motor-traffic did not yet justify the immediate wholesale remodelling of the macadam roads of the country; a beginning must be made somewhere, and no doubt the difficulty, which was mainly one of cost, would be overcome gradually. Mr. Perry regretted that he could not share the Author's enthusiasms, nor endorse his estimate of cost. In the estimate an important item was omitted, namely, the cost of land and of compensation of various kinds. The assumption that the land would be given free for the public good was not a practical one; and the further assumption that there was everywhere a fence which could be moved for 1*s*. per yard, was enthusiastic. What about widening in cuttings and filling; the alteration of bridges and culverts? Having regard to the fact that cottages with gardens, etc., gathered along the sides of public roads, it would probably be found in most cases that it was cheaper to make a new road than to widen an existing road. It might be thought that the operation of 14 and 15 Victoria, cap. 92, had kept a space 60 feet wide; but it had not, as anyone could see. Mr. Perry had just been instructed by his county council to abandon a prosecution for infringement of this law; and recently he had come across a case in which a county councillor had just built himself a new range of farm-buildings on the face-line of the fence of a narrow public road. The proposed arrangement of a light road and a heavy road was unworkable; who was to see that each road was kept for its particular traffic? There was something to be said for a fenced footpath along the centre, for giving wider passing-room than there was at present, and,

possibly, for separating the up from the down traffic. In respect of the dust from pneumatic tires no thin-skin treatment could be successful. A wire fence along the centre of a public road with standards 9 feet apart was surely not good enough for its position and duty. Mr. Perry.

Mr. J. F. J. REYNOLDS observed that, to the advantages which Mr. Jebb claimed would result were the repair of all county highways to be undertaken by the respective county councils, might be added that a more efficient and better-trained staff would be collected, which would result in economy of labour, and at the same time would afford more regular employment. The class of men now engaged upon the repair and maintenance of country roads was hardly equal to requirements. An authority having a large mileage of roads to maintain would be in a better position to make contracts for materials, and might even acquire quarries and undertake its own transport of materials, at a considerable saving on the present cost. And all these advantages would probably be further increased were the powers vested in one central authority to supervise all country roads, with jurisdiction over all waste lands, fences, and hedges bordering upon roads, and under bridges. Mr. Blackwall's suggestion for twin roads had many excellent points, especially as such roads could, in some cases, be constructed without the purchase or grant of additional land, if the existing waste strips were made use of. They should have the further advantage of providing the required additional road-surface without the introduction of central gutters, which were likely to render the roads dangerous to users of rubber tires, and might tend to soften the road-surface, if the drainage was not thoroughly effective. The suggested width of 46 feet between banks, however, appeared to be excessive (except where a very large number of vehicles had to be provided for), as it would be sufficient for three vehicles abreast on each half of the road. The present metalled width of the majority of country roads did not exceed 15 feet, and as there was usually an extra foot or so to spare on each side of the metalled surface, this width was found to be ample to allow of two vehicles passing, or of a fast vehicle overtaking a slow or standing vehicle, without reduction of speed. Probably a very large mileage of twin roads could be constructed in some counties with little outlay beyond that for actual works. In other parts of the country, however, there were already more roads than the traffic required, and as an alternative to twin roads in some cases it might be possible, where two existing roads were nearly parallel, to limit one to heavy traffic and the other to light, thus retaining the advantages of the twin system without Mr. Reynolds.

Mr. Reynolds. increasing the actual area of roadway to be maintained within that district. An important point for consideration in the construction of any system of roads for fast traffic was the crossing of other roads on the level, and in order that this might be done with as little danger and reduction in speed as possible, all corners would have to be constructed so as to allow crossing traffic to be visible for as long as possible, and notice-boards should be erected showing clearly which line of traffic had to give way to the other. Apart from the construction of new and special roads, it was evident that much might be done to improve the existing country roads for all classes of traffic, more particularly in the way of straightening, diverting, regrading, etc. Now that the construction of foundations to roadways was so much better understood, there was no longer any necessity for roads to follow hard or high ground. It would be a great advantage if new fences were of wire or some other kind of open construction, so as to allow road-surfaces to dry quickly and the summer dust to spread itself over the adjoining fields. Shade trees of suitable kinds, and judiciously planted so as not to interfere with the drying action of sun and air, would not be objected to. Some years must necessarily elapse before the respective merits of recent improved methods of laying road-surfaces could be fully proved. Any method of construction, or material, to be successful, should produce a surface as hard as possible and free from any ingredient which might cause it to become dangerous under any conditions of weather. The plea that a road-surface was destructive to tires should not be allowed to carry weight, provided it was satisfactory in other respects, for it was only just that the owner of a vehicle should pay in proportion to the wear of his tires, rather than that the road-surface should suffer. Care should be taken, when testing new methods, that a sufficient length was laid over a characteristic section of the road, for the mud from other portions of the road was often carried by wheel-tires for a considerable distance along the trial length, and might entirely alter its behaviour under certain conditions of weather. The effect of all kinds of weather upon the trial length should be carefully noted, *e.g.*, its ability to dry quickly after rain and snow, its resistance to the action of frost, etc.

Mr. Saner. Mr. J. A. Saner was glad that Mr. Jebb had brought forward the question of the consolidation of the management of the country roads. The provision of easy, safe, and relatively rapid means of communication between distant points, whether by railway, waterway, or roadway, was one of the most important problems engineers had to face at the present time; and he heartily supported the Author

in his suggestion to place the control of the country roads in the hands of fewer authorities. The main roads of Cheshire, so far as they were under the direct control of the County Council, were excellent, and as they had an aggregate length of 511 miles the main routes of the county were well served. There were, however, certain lengths, repairable by either railway- or canal-companies, which were easily distinguished by their inferiority, and which should certainly be placed under the county, with power to levy the cost of their proper maintenance. If the rural roads were also placed under the County Council the mileage would be greatly increased, but it was certain that great economy, more especially in supervision, would ensue. The highly-trained district surveyors, mounted on motor-cycles or cars, would be able to supervise many more miles; and a few additional steam-rollers and road-gangs of thoroughly efficient men would easily replace the numerous small and less efficient gangs now engaged by the rural authorities. Probably the most difficult problem to deal with was that of reducing the dust in summer, and the mud in winter; and he thought a good step would be made in this direction by using less of the muddy so-called binding-material. In a short length of road made by him about 6 years ago he refused to allow anything but clean grit to be used; and although the contractor objected strongly and declared that the road would never set, it had proved to be one of the cleanest and hardest roads in the neighbourhood. Whether the result was due entirely to the use of clean grit and sand for the binding-material, or whether it was due partly to the very good foundation which was also put in, was difficult to say; but probably both had contributed, a good foundation and good drainage being essential to a good road. Another source of economy would be in the repair of the numerous culverts and bridges under the roads: the Cheshire County Council were now more or less responsible for upwards of three hundred bridges, and it would certainly not increase their responsibilities unduly if the remaining bridges of the county, which were mostly mere culverts, were placed under the supervision of their staff. One important point he wished to put forward was that railway and other companies should not be allowed to divert straight roads when building elevated bridges across their railways or canals. He did not blame the companies for taking the easiest and cheapest course, if Parliament sanctioned it; but it was important now to do away with as many bends and corners in roads as possible, and he thought that where such bridges were necessary they should be built in the line of the road and not alongside or some little distance away. For example, the new bridges built by the North Eastern Railway Mr. Saner.

Mr. Saner. Company at Saddlethorpe, in East Yorkshire, and by the London and North Western Railway Company at Atherstone, both caused breaks in what were otherwise perfectly straight roads; the latter broke the continuity of Watling Street, and replaced a straight road with one in which there were four sharp bends. In most cases there should be no difficulty in temporarily diverting the road-traffic and building the bridge on the site of the existing roadway; and when the cost of doing this, owing to houses or other obstacles, was estimated to be greatly in excess of the present practice, leave to divert should be granted only after full local inquiry. He regretted that he must disagree entirely with Mr. Blackwall's suggestion of a double roadway. In the very places where a double road would be of use, namely, villages and small towns, it was impracticable, and it would be much better to spend any available money in consolidating and widening the existing roads.

Mr. Thropp. Mr. James Thropp contributed the following notes of his experience of road-management in Lincolnshire. In the Parts of Lindsey there had been marked progressive improvement of main roads and highways in nearly every district, but chiefly in those districts where the steam-roller had been in use. In some rural and urban districts in the county the steam-roller was not used, and there the state of the roads was dependent upon the skill of road-men in the use of the material provided, and their care in sweeping, scavenging, and giving constant attention to the surface drainage. The difficulties of securing good roads throughout were considerable, because of the diversity of ideas of the district councils. Where the district council appointed a district surveyor with complete control over his men, there would be an approach to uniformity of methods of repair; but where the surveyor only inspected the roads, which were in charge of sub-surveyors who paid and engaged the road-men, and each sub-surveyor had his own ideas of reparation, then waste of material and money ensued, and the roads did not improve. So long as the county and district councils were free to adopt any method of management they pleased, diversity, instead of uniformity, must necessarily be the result. When steam-rolling on county roads, the scarifier was sometimes used, and $2\frac{1}{4}$-inch granite was spread in the middle, and slag at the sides, with a fall of about $4\frac{1}{2}$ inches between the centre and the sides. In some districts granite alone was used when steam-rolling. Long lengths of steam-rolling could not be afforded without unduly raising the cost per mile. The roads which were not steam-rolled were first middled in sections of 30 yards, missing 90 yards; the width varied between 3 and 7 feet, and the ends of each long patch were tapered. After the middlings had been

laid with all the stones touching, patching with small patches, 3 feet by 4 feet, was proceeded with, and as few patches as possible were put down—just enough to keep the traffic on the move, and to prevent tracking. By this plan, in 4 years the road was middled the whole length. The road-men were directed to fill up low places as they occurred with very small granite, and also to sweep and scavenge frequently. In the spring and late summer siding was done with a line and ritter, and then the surface-drainage had the attention of the road-men. It was not possible to water county roads; if it were, with efficient sweeping there would not be much annoyance from dust. In Lindsey there was no local material suitable for road-maintenance, and therefore English and Belgian granite and whinstone were used for main and other important roads, and for ordinary highways slag was procured from either the Frodingham or the Leicestershire furnaces. For the improvement of main roads in urban districts, where heavy traffic would justify the outlay, tar-macadam was proving of great service, and would be more used in the future as its value became known to district and county councils. This material, although costing about three times as much as granite, would probably actually cost less when the life of a "tarmac" road had been ascertained. A short length, laid down 2 years ago by Mr. Hooley, County Surveyor of Nottinghamshire, at Newark, where exceedingly heavy traffic from four malt-kilns passed over it, showed no signs of wear at present. Messrs. Parry and Sons, of Kirton-in-Lindsey, had built a plant for breaking the blue lias limestone obtained from their quarries and heating and treating it with boiled tar; this, he thought, was quite equal to the best tarmac made with slag. A recent experiment which was being watched with great interest, had been made by the Gainsborough Rural District Council with tarmac on a district road in the county of Lindsey, an area of 900 yards being laid; and, in order to compare results, a length, having an area of 2,305 square yards, of Messrs. Parry's tar-macadam, had been laid in continuation of Mr. Hooley's material. The details of cost of these two kinds of surface, including carriage, carting, steam-rolling and labour, were:— Mr. Thropp.

"Tarmac" (Area covered 900 Yards).

	£	s.	d.
101 tons 1 cwt. at 13*s*. 6*d*.	68	4	3
8 „ 14 „ „ 7*s*. 6*d*.	3	5	3
Hauling at 2*s*. 6*d*. per ton	13	15	0
Rolling and spreading	10	0	0
	95	4	6

Cost per yard 2*s*. 1*d*.

Mr. Thropp.

TAR-MACADAM (AREA COVERED 2,305 YARDS).

	£	s.	d.
213 tons at 10s. 6d.	111	16	6
73 „ „ 12s. 6d.	45	12	6
Hauling „ 2s. 6d.	35	15	0
Rolling and spreading	20	0	0
	213	4	0

Cost per yard 1s. 10d.

A great advantage in the use of tarmac instead of granite was that there was no necessity to water the surface of the road when steam-rolling. Steam-rolling with granite and slag at the sides cost in 1904 about 7·21*d.* per square yard. The following were the details of the cost of steam-rolling recently executed with whinstone on the Lincoln and Brigg Road:—

AREA COVERED 4,180 SQUARE YARDS.

	£	s.	d.
201 tons 19 cwt. whinstone at 12s. 2d.	122	17	1
Hauling whinstone at 3s. 8d. per ton	37	0	6
Hire of roller	17	10	0
Labour and spreading	8	2	6
Leading water	4	1	0
	189	11	1

Cost per yard 10·88*d.*

He feared that Mr. Blackwall's idea of subdividing existing roads would be too expensive and would cause much friction between the various local authorities. Further, his estimate of £32 per mile for the maintenance of main roads was too low. The cost of rural main roads in Lindsey during the years 1896–1904 varied between £54·18 and £66·98 per mile, and that of urban main roads between £113·19 and £157·09 per mile.[1] The cost of the ordinary highways in Lindsey during the year ended 31st March, 1904, was £26·72 per mile.

Mr. Waring. Mr. F. J. WARING, having no professional connection with the maintenance of roads in the United Kingdom, could offer no opinion as to whether the introduction of motor-driven vehicles and bicycles had so increased the traffic on some of them as to demand, either now or in the near future, the provision of additional accommodation for that class of traffic; but, assuming this to be the case, he thought that the proposal made by Mr. Blackwall, though it required elaboration and modification in many details, and would probably be much more costly than it was estimated to be, was worthy of consideration. But under Mr. Blackwall's suggested

[1] J. Thropp, "Statement of Costs of Main Roads (Lincolnshire, Lindsey) for the year ended 31st March, 1904." Louth, 1905.

classification of the traffic, motor-omnibuses like those now plying in London (which carried 34 passengers besides conductor and driver, and weighed when loaded about 7½ tons), ordinary omnibuses carrying 26 passengers besides conductor and driver, and pair-horse railway collecting-vans weighing when loaded 6½ tons, which were often driven at a trot, would all use the light road; and Mr. Waring had no hesitation in coming to the conclusion that if this road were constructed without a proper foundation, as was apparently intended, it would be quite unequal to carrying this class of traffic. The suggestion that in towns and villages the twin roads should unite, which presumably would also apply at bridges either over or under the road, seemed objectionable, because not only were the roads generally narrower in villages and towns than in the open country, but traffic was generally greater; and he would suggest as an alternative in such localities, if funds were available, the construction of a new road for the fast traffic, passing round the village or town. As regarded the bridges, any constriction of traffic at them was also objectionable, for in order to obtain square crossings, or to reduce the angle of the skew, and also to obtain the desired headway, such bridges were often approached by sharp curves and steep gradients; at such places the construction of new bridges or the widening of the existing ones would be desirable. He could not agree with Mr. Blackwall's proposals respecting the drainage of the twin roads, as it seemed to him to be a mistake to concentrate this under the footpath more than was necessary, while the cross gutters under the heavy road would be objectionable. As an alternative Mr. Waring would suggest side ditches outside both the heavy and light roads, which would carry the drainage from the outer halves of each, while that from the inner halves might be dealt with by the pipe drain under the path, discharging, like the side ditches, into the first stream passing under the road. Further, the camber shown in *Fig. 1* was, he thought, excessive, as it should be only enough to allow the water to run off freely. Instead of a profile curved throughout, he would suggest one consisting of two straight lines, falling, at the minimum angle required to ensure drainage, from the centre, where they would be connected by a short, flat curve; the formation should be of the same profile as the surface, thus securing uniform depth of foundation and metalling throughout. A 6-inch foundation of stones carefully packed and rammed, covered with 6 inches of hard stone metalling, broken to pass a 1½-inch ring, well rammed, covered with a thin layer of clean quartz or other gravel, and then well watered and rolled, had, in his colonial experience, given very satisfactory results. Every road Mr. Waring.

Mr. Waring. should be waterproof, and if "tarmac" or other combination of tar and macadam would, as he thought, probably aid in securing this essential feature, he would advocate its use.

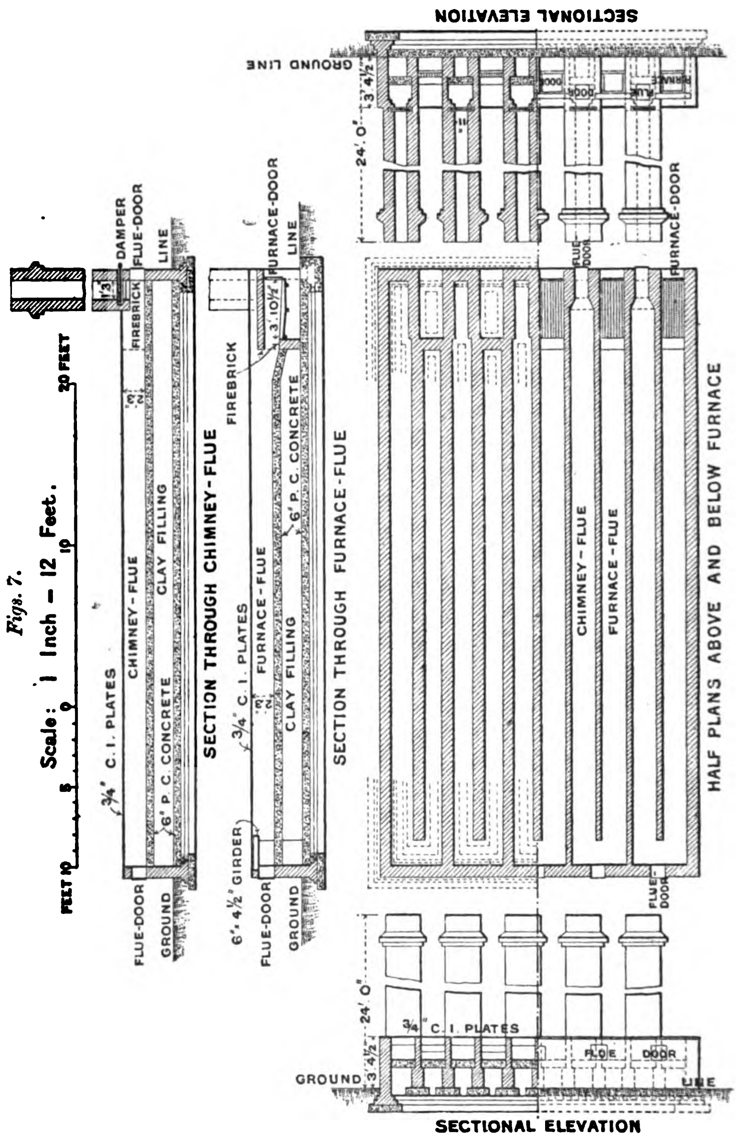
Mr. Whyatt. Mr. H. GILBERT WHYATT communicated the following results of his experience of maintaining and improving roads. First, with regard to material, if local stone could be obtained cheaply, the local authority would always succumb to the temptation of using it; but if no local stone was available, and the stone used had to be brought long distances, the truest economy was to pay a somewhat high price for a tough and durable material. The only stones which came within this category were granite, syenite and basalt. In the final selection a stone should be adopted which contained a minimum of mica and felspar, for the latter was the constituent which on disintegration formed kaolin or clay, and a stone containing a large proportion of it was less valuable for road-making than a stone from which it was almost or entirely absent. On taking up his present appointment as Borough Engineer of Grimsby in 1900, he had found that about one-half of the highways were coated with a granite containing a large proportion of mica and felspar, and owing to disintegration of the stone under the influence of the weather the quantity of mud to be dealt with in wet weather was appalling. It was clear that some material other than such granite must be adopted, and his attention was called to a short length of street which his predecessor had coated with basalt. He thereupon laid down the same material in two other lengths of street, and the experiment was so satisfactory that little difficulty was experienced in obtaining the consent of the Highways Committee to the adoption of columnar basalt from Linz-on-Rhine, with excellent results. The method adopted for the repair of streets was the following:—The surface was first scarified, and the whole of the surface material was removed down to the foundation; on this a 4-inch coating of basalt was laid, and rolled until fairly consolidated. If the material taken from the street was suitable for binding, it was then spread over the basalt together with a sufficiency of selected road-sweepings. A water-cart was used, and the road was then rolled again until the basalt and the binding were thoroughly intermixed. The road was left for 2 or 3 days to dry; the roller was again passed over it, and it was left in perfect condition. The sweeping of the streets was generally done in two operations. Some streets were swept daily, some every 2 days, and some twice a week. At the first operation all paper, straw, horse-dung and similar matters were removed and tipped to waste. The second operation took place only in wet weather, when the actual mud was removed and tipped in

depots. Here it was allowed to stand for some months, that was, until organic matter had disappeared; and the remaining portion, consisting almost entirely of mineral matter, was used, as above described, as binding material when repairing streets. One result of the adoption of basalt was that not sufficient mud was swept from the streets to furnish the quantity required for repairs during the year. The actual cost of new roads in two separate parts of Grimsby was— Mr. Whyatt.

		Road A. s. d.	Road B. s. d.
Excavation	per square yard	0 4½	0 6
Chalk foundation 12 inches thick	„ „ „	1 10	2 0
Basalt 4 inches thick	„ „ „	2 6	2 2⅜
		4 8½	4 8⅜

These two roads had been carried out by separate contractors—hence the difference in the cost of each item; but the total cost per square yard in the two cases was remarkably close. The cost of repairs as carried out by Mr. Whyatt was 2*s.* 4*d.* per square yard, which was nearly the average of the prices charged for basalt by the two contractors. The annual cost varied according to the amount and weight of traffic: some roads cost 2*s.* 4*d.* per square yard annually; others lasted for 2 or 3 years; and some looked as though they might last 10 years, or even more. The total cost of roadwork in the borough of Grimsby was about £14,400 per annum for a length of 45 miles (some of which was country road), giving a cost of £320 per mile, including footpaths, watering, and sweeping, but not sewers. In order to maintain good roads it was necessary that the contour should be well preserved, so that no water might stand upon the road at any place. Horse-droppings should be moved as early as possible, so that they might not be ground into dust by passing vehicles; by this means the nuisance from dust was minimized. The roads should by all means be kept dry; one road in the Grimsby district running east and west was seldom dry and soon wore into mud, while roads running north and south were hard and dry. Scraping of the roads should be prohibited as far as possible. Tarred macadam was no new material, having been made in Nottingham as far back as 1840, and it had been used by Mr. Whyatt for some years. He had lately constructed a hot-plate (*Figs.* 7) for the making of tarred macadam, and hoped in the near future to use it extensively. The following was the method of mixing and laying he had adopted, but the cost could not yet be given, either of the laying or of the maintenance. Slag was spread over the whole area of cast-iron plates to a thickness of 7 to 10 inches (the best thickness would be ascertained in the course of a few months' working). If the slag

Mr. Whyatt. was very wet, it might be necessary to turn it over on the hot-plates. Fires were kept burning day and night, and the stone was left on



Figs. 7.

Scale: 1 Inch = 12 Feet.

the hot-plate until it was quite dry and warmed through. A mixing-board was provided, about 10 feet by 4 feet, and the warm slag was

wheeled to this board as required. About three barrowfuls were assembled, and over this a sufficient quantity of tar composition was poured, the heap being then turned over until every stone was covered with tar. It was next placed in a wheelbarrow, wheeled along the elevated plank and tipped into the heap. The tar composition consisted of 12 gallons of tar, ½ cwt. of pitch, and 2 gallons of creosote: these quantities sufficed for 1 ton of slag. The whole was boiled to a proper consistency, which occupied about 1¼ hour, and was kept well stirred while coming to the boiling-point. The whole of the mixing was done while the stone was warm, and the heap was left to mature for a period of 6 to 9 weeks. Care must be taken, however, that the stone was not too hot, otherwise it would burn all the nature out of the tar. Supplies of slag and slag screenings should be arranged so that the installation was never idle. The slag should be of the three following gauges: 2½ inches, 1 inch, and ½ inch or ⅜ inch. Slag full of sulphur must not be used. Five able-bodied men were employed; one leading hand, two wheelers, and two mixers. The camber to form the crown of the road was only about one-half of that required for ordinary macadam, and might be reckoned at about ⅜ inch per foot. The finished macadam should be 5 inches thick in the centre of the road and 3 inches thick at the channel; and the foundation must be arranged so as to give this thickness. The foundation must be well rolled and consolidated Mr. Whyatt.

Size of Macadam.	Thickness in Centre.		Thickness at Channels.	
	Ins.	Ins.	Ins.	Ins.
2½-inch	4½ thick rolled down to	3½	3 thick rolled down to	2
1-inch	2 ,, ,,	1	1 ,, ,,	½
½-inch or ⅜-inch . . .	1 ,, ,,	½	1 ,, ,,	½
Total . .	7½	5	5	3

before any tarred macadam was laid, and any low places must be made up with slag, chalk, or other suitable material. The work should be done in dry weather, and no water should be used under any circumstances. On completion, the surface should be sprinkled with fine sand or fine screenings. In certain states of the weather the surface of a tarred macadam road became very slippery and dangerous for horses, and a considerable stock of slag screenings or other suitable material must be kept at hand for sprinkling the surface. Tarred macadam would certainly reduce the quantity of dust to a very small amount, but during the period of transition it would appear to be quite as bad as ordinary macadam, owing to dirt

G 2

Mr. Whyatt. being brought on to it from adjoining lengths of the old-fashioned roads. Mr. Whyatt had not the slightest faith in any of the dust-preventives so much advocated at the present day. It did not seem reasonable that a coating of tarred macadam $\frac{1}{16}$ to $\frac{1}{8}$ inch thick should be substantial enough to withstand the ordinary traffic on a road; and although such experiments promised well for the first few weeks, they were bound to result in failure. Of the arguments brought forward by Mr. Jebb in support of his contention that all country highways should be kept in repair by the county councils, Nos. 1 and 3 were the weightiest. One of the reasons for the poor attention paid to country roads by the rural districts and smaller boroughs at the present time was that they were afraid of not having the cost reimbursed to them by the county councils. If they could be assured on this point, country roads could be and would be much improved.

Mr. Blackwall's suggestion for two roads side by side, as an engineering question discussed from an academic point of view, was well worthy of consideration; but it was not likely that rural districts would ever tax themselves to provide good roads on which non-ratepayers might disport themselves; and it would be an impossible task to convince them that the construction of such a road should be paid for by anybody but motorists. He considered Mr. Blackwall's estimate of 20*s.* per lineal yard 16 feet wide (the fencing items omitted), or 3*s.* 9*d.* per square yard, somewhat low; but the Author admitted that the data for these estimates were unreliable, and for the same reason other figures were not suggested, though, as stated above, Mr. Whyatt found that similar work within 1½ mile of a railway cost 4*s.* 8½*d.* per square yard.

Mr. Jebb. Mr. Jebb, in reply, was glad to find that his view that the country roads should be improved and maintained on more scientific principles, and that the road-authorities should be reduced in number, was unanimously supported. He was in entire agreement with Mr. Lynam and others as to the most important principles of road-construction, namely, a good foundation, perfect drainage, the best obtainable material for macadam, and nothing but clean sharp chippings used for binding. The experience of Mr. Whyatt and others of the use of tar-macadam was instructive and encouraging. With reference to the opinion of Mr. Dryland and many others that the real difficulty in the way of making any great improvement in country roads was want of money, he wished to express strongly his opinion that the time had come when this difficulty ought to be met and grappled with. To go on as at present—merely keeping the roads passablo at extravagant cost (as was the case in many counties)

—was wanton waste of the ratepayers' money. He ventured to make Mr. Jebb the following suggestion:—That, after all country roads should have been transferred to county councils, the county surveyors should be instructed to draw up a carefully-considered scheme for the improvement of such roads as needed improvement, to make an estimate of the cost, to consider which roads should be first dealt with, and to fix a time for the completion of the whole. When the county councils had this information, they should be empowered to borrow, and the State to lend, at a low rate of interest, such sums as might be necessary for putting the roads in order, and the repayment should be spread over 30 to 40 years. The highway-rate should be fixed at a reasonable amount, and not be varied until the loan had been repaid.

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27 February, 1906.

Sir ALEXANDER RICHARDSON BINNIE, President, in the Chair.

The discussion upon the Papers by Messrs. Jebb and Blackwall on Country Roads occupied the evening.

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6 March, 1906.

Sir ALEXANDER RICHARDSON BINNIE, President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

JOHN SIDNEY ALFORD.
ERNEST CRANSTON GIVEN.
ALBERT DANIEL GREATOREX.
DAVID BAYNE HORN, C.I.E., F.C.H.
ARTHUR JOHN MARTIN.
WILLIAM RICHARDS WILLIAMS, F.C.H.

And had admitted as

Students.

CHARLES WALTER MAYBERY BEST, B.A. (*Oxon.*)
GEOFFREY DRAKE BROCKMAN.
ALEXANDER JOSEPH COHEN.
ALBERT CROAD.
FRANK ALPIN EAST.
WALTER HENRY EDRIDGE.
THOMAS PERCIVAL GEEN.
ARTHUR JAMES HILLMAN.
HOLIDAY LEYCESTER HUNTER, B.Sc. (*Manchester*).
ARCHIBALD MUIR HUTCHINSON.
LAWRENCE WATSON JAMESON.
HARRY CHICKALL LOTT.
EDWARD LOVELL MANLEY.
HORACE ABRAHAM MYERS.
JOHN ARTHUR TOMLINSON.
ALFRED VIPAN.
JOHN FRANCIS EDWARD GELSON WALKER, B.A. (*Cantab.*)

The Scrutineers reported that the following Candidates had been duly elected as

Members.

JOSEPH ARTHUR LOVEL DEARLOVE.
HARRY DENT.
THOMAS WILLIAM ROBERT HUGHES.
OSWALD CHRISTIAN ORMSBY.
THOMAS HALE WOODROFFE.

Associate Members.

FREDERICK GUY TREVENEN ADAMS, Stud. Inst. C.E.
ERNEST WITTON BOOTH.
FRANCIS GRANT CARRON, Stud. Inst. C.E.
HERBERT ROSTRON DISLEY, Stud. Inst. C.E.
JOSEPH POLLARD HODGSON, Stud. Inst. C.E.
GEORGE EDWARD HUNT, Stud. Inst. C.E.

Associate Members—continued.

JOHN EDWARD LISTER, Stud. Inst. C.E.
THOMAS HERBERT MINSHALL.
CHARLES NOEL MOBERLY, Stud. Inst. C.E.
ARTHUR GEORGE FRITZ NAPIER, M.Sc. (*Victoria*), Stud. Inst. C.E.
LEONARD LESLIE ROBINSON.
WILLIAM PERCIVAL ROBINSON.
FRITZ SCHOEBERLEIN, Stud. Inst. C.E.
LESLIE MOFFAT SETH-SMITH, B.A. (*Cantab.*)

Associate.

ROBERT STANSER TEMPLETON.

(*Paper No. 3534.*)

"The Widnes and Runcorn Transporter-Bridge."

By JOHN JAMES WEBSTER, M. Inst. C.E.

THE River Mersey, flowing from north to south past Liverpool, separates the busy and populous districts of South Lancashire from those in Cheshire; and as there were no means of dealing with vehicular traffic from one county to the other, except the cumbersome ferry-boats at Liverpool, or the narrow bridge at Warrington, about 18 miles up the river, considerable inconvenience was experienced by the manufacturers, farmers, and others on both sides of the river. There is a public footpath on the London and North Western Railway Company's bridge which crosses the river at Runcorn, but, as this is approached by a long steep gradient on one side and by a long flight of steps on the other, it affords but unsatisfactory accommodation for foot-passengers only. So long ago as the beginning of the nineteenth century, when the population of the various busy centres was hardly a tithe of what it is at present, a strong project was set on foot for building a roadway bridge across the river at Runcorn; and in the year 1817 Telford was asked by an influential committee to report upon a suitable scheme. At that time Captain Browne, of the Deptford Chain Cable Works, had designed what was really the first chain suspension-bridge in this country, and, working in conjunction with Telford, he made a model of a bridge of 100 feet span upon this principle, which, when completed, was safely driven over by Telford and others in a hackney carriage, amid the cheers of an excited and interested crowd. The practicability of this system being thus established, Telford designed a bridge on this principle for Runcorn,[1] but although the committee

[1] See "Life of Thomas Telford," p. 219. London, 1838. The bridge was to have a span of 1,000 feet between the centres of two stone pyramidal towers, a 20-foot roadway, the clear waterway at the centre of the span being 70 feet from high-water

reported favourably upon the scheme, and subscribed liberally to a guarantee fund, nothing further was done in the matter. This scheme, however, was the forerunner of Telford's well-known suspension-bridges at Conway and the Menai Straits.[1]

Several schemes have been proposed since that time for crossing the river at Runcorn, by means of either a bridge or a tunnel; but until the transporter-bridge described in this Paper was projected, the cost of the proposed works in each case had prevented the schemes from being brought to a successful issue.

Application was made to Parliament for power to construct a transporter-bridge between Widnes and Runcorn, and although there were eleven petitions against the Bill, it passed through both Houses, and received the Royal Assent in July, 1900. A company was formed, with Sir John T. Brunner, Bart., M.P., as Chairman, and the necessary capital having been subscribed, the contract for the construction of the bridge was let in November, 1901.

General Description.

The bridge is of the transporter type, and although it is the first of the kind to be erected in England, similar bridges are working successfully at Portugalete, Bizerta, Rouen and Rochefort, and others are in course of construction at Newport (Monmouthshire) and elsewhere. The principle of the bridge was designed by Mr. Charles Smith, of Hartlepool, who planned one of this type for crossing the River Tees at Middlesbrough in 1873;[2] this was favourably reported upon by Sir Benjamin Baker, Past-President Inst. C.E., but, owing to lack of funds, it was never constructed.

The Widnes and Runcorn bridge crosses the River Mersey and the Manchester Ship-Canal with one span of 1,000 feet between the centres of the towers, four approach-spans of 55 feet 6 inches each on the Widnes side, and one span on the Runcorn side, the total span from shore to shore being 1,305 feet. The roadways are 35 feet wide between the parapets, with footpaths 6 feet wide on each side, which are widened out to 70 feet next the towers, for marshalling the traffic and for the erection of waiting-rooms.

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level to the underside of the roadway. The estimated cost for the structure alone was £90,490. Telford made a model of the bridge to a scale of 20 feet to 1 inch, and this interesting memento is still in existence in a building belonging to the Shropshire Union Canals at Ellesmere, Shropshire.

[1] For an interesting account of early suspension-bridges constructed in America see *Engineering News*, vol. liii, p. 269.

[2] *Engineering*, vol. xvi, p. 60.

The general elevation, plan, and cross section of the bridge are shown in Figs. 1, Plate 1. In construction the bridge is similar to an ordinary cable suspension-bridge with stiffening-girders, with the exception that, instead of the costly high-level approaches, the traffic is taken along low-level approaches on to a car suspended from a trolley running on the underside of the stiffening-girders, and this car is propelled electrically from one side of the river to the other. The height from high-water level to the underside of the girders of the London and North Western Railway bridge, which is lower down the river, is only 75 feet, and this fact determined the height to the underside of the girders of the transporter-bridge; but to meet the wishes of the Board of Trade it was made 82 feet. The two towers on the Widnes side are built on the foreshore of the river, and the two on the Runcorn side are built in the Ship-Canal, the bed of which at this point was originally the Cheshire foreshore of the river. Each tower supports, on cast-iron saddles and rollers, a steel-wire cable, from which is suspended the stiffening-girder; upon the bottom flange of the latter runs the trolley, from which the car is hung. The stiffening-girders project beyond the sides of the towers, to enable the car to run into a dock between the towers. The car is 55 feet long by 24 feet 6 inches wide, and its deck is open, except over a portion on one side, which is occupied by a closed shelter. The electric motors for propelling the car are fixed to the trolley, but are under the control of the driver in the cabin at the top of the car, from which an uninterrupted view of the traffic on the river can be obtained.

As there was no electric power available within a reasonable distance on either side of the river, a generating-station was built at the bottom of one of the towers on the Widnes side, which supplies the necessary current for driving the machinery and illuminating the structure.

Although there are several fine examples of cable suspension-bridges in America, this is the first of any magnitude to be built in England; and in designing it several very interesting problems presented themselves, which will be referred to in the following more detailed description of the various parts of the structure.

Approaches.

On the Runcorn side, a new approach-road, having an average width of 35 feet, was constructed from Church Street to the towers, a distance of about 470 feet. For 415 feet the road consists of retaining-walls of Portland-cement concrete with sandstone faces, filled in between; the roadway being of granite macadam, and the

footways of granolithic pavement. Where the roadway widens to 70 feet, next to the towers, the approach road, being over the Manchester Ship-Canal, is built on curved steel braced girders as shown in the general elevation (Figs. 1, Plate 1). These girders are of various spans, to allow for the opening forming the car-dock. The roadway crosses the double line of railway-sidings belonging to the Ship-Canal Company, and is here constructed of steel corrugated flooring-plate, 12 inches deep, spanning the whole width. A stone parapet-wall, 3 feet 9 inches high, is built on each side, with pedestals at intervals for carrying the lamp-standards.

The approach on the Widnes side is 320 feet long, and as it is built mainly on the foreshore of the river, it consists of four spans of continuous girders of 55 feet 6 inches span, bolted to cast-iron columns, 14 inches in diameter, fixed to the rock; the remaining length on the shore is constructed of retaining-walls and earth filling, the roadway being paved with wood blocks, and the foot-paths with granolithic pavement. A subway 10 feet wide is built under the approach-road, to give access to the slipway, and is lined throughout with white glazed bricks. An ornamental iron hand-railing is fixed on both sides of the viaduct-approaches, stone parapet-walls surmounting the retaining-walls.

A stone toll-house is built outside each of the approaches, the doorway opening on to the footways; and a 10-ton platform weighing-machine is placed outside in the roadway, the recording levers being inside the toll-house.

Foundations of Towers.

Borings were taken at the sites of the proposed towers and anchorages, and the strata are indicated in Figs. 1, Plate 1. On the Widnes side the rock crops out at the surface and is covered with a layer of mud averaging about 12 inches in thickness. On the Runcorn side rock is reached at an average depth of 20 feet below Ordnance datum, and it crops out at the surface near the anchorages. The rock is hard red sandstone on the Widnes side, and yellow sandstone of a softer nature on the Runcorn side of the river. The stone on the Widnes side was very rotten and loose at the surface; this was cut away until solid rock was reached, upon which the tower-cylinders were founded. In cutting out this bad rock, at a depth of about 3 feet from the surface, a curious stratum was found, of a substance having the consistency of white lead, of a dirty yellow colour, and about 4 inches thick; it did not occur in pockets, but extended over a considerable area.

Each tower rests upon four cast-iron cylinders 9 feet in diameter (Figs. 2 and 3, Plate 1); the cylinders are placed 30 feet apart between centres, and the centres of the two towers are 70 feet apart. The cylinders were cast in complete rings 4 feet high, the thickness of metal being 1⅛ inch; the internal flanges were well bracketed, and had turned faces with a shallow spigot-and-faucet joint. The top lengths of the cylinders taper to a diameter of 6 feet 6 inches at the top, the level of the tops of all the cylinders being 30·75 feet above Ordnance datum. The bottom lengths of the cylinders on the Widnes side are bolted to the rock with long lewis-bolts 2 inches in diameter. The cylinders were filled with Portland-cement concrete mixed in the proportion of 7 to 1, the holding-down bolts and washer-plates for the towers being built in as the work proceeded. The cylinders were braced together by strong steel girders with curved bottom flanges, and by round diagonal tie-rods and channel-bar struts; the girders being 9 feet deep next the cylinders, and 6 feet 6 inches at the centre. As the bed of the river at the site of the Widnes towers was dry at low water, there was no difficulty in fixing the cylinders; but the Runcorn cylinders, being in the Manchester Ship-Canal, had to be sunk by means of compressed air, an operation which will be described when the methods of erection are considered.

Tower-Fenders.

Timber fenders are built around the tower-cylinders, consisting of greenheart piles, 12 inches square, driven 10 feet apart, with raking struts at the back. The piles are braced together; and at the front, creosoted pitch-pine fenders 12 inches by 6 inches, faced with half-round bar iron, 5 inches by 1½ inch and spaced 3 feet between centres, are bolted to the piles. There is a space of 9 inches between the piles and the ironwork, giving ample clearance in the event of a vessel running into the fenders.

Cable-Anchorages.

Simultaneously with the erection of the tower-cylinders, the cable-anchorages were constructed, as shown in Figs. 4, Plate 1. The rock was excavated to the depth and form shown, and the cast-iron anchor-plates were placed in position at the bottom; each plate receiving two forged-steel links, 5 inches by 4 inches, secured with a cotter at the back of the plates. The links are alternately single and double, increasing in dimensions as they approach the top,

where they are set at varying angles on cast-iron pillows, until the last link has a fair lead to the cable-backstays; the cast-iron pillows and the top links rest on stone embedded in concrete. The top links, which are 6½ inches deep by 4 inches wide, are attached to three steel cross-heads, which receive the adjusting-screws of the cable-backstays, described later on with the cables.

On top of the anchor-plates were placed two rows of steel joists 8 inches by 4 inches and weighing 19 lbs. per foot, 20 feet in length transversely and 9 feet longitudinally; and when these joists were in position, the bottom of the anchorage-pits was filled with cement concrete mixed 7 to 1, the concrete being well rammed under the anchor-plates and the cross joists.

In most suspension-bridges of any magnitude, it has been the custom to leave the links or cables in the anchor-pits accessible for inspection. It will be found, however, that in most cases the inspection has not taken place, and that the links or cables are badly corroded; and in many bridges it is impossible to paint the underside of the links. It was therefore decided, for the foregoing reasons and also because there was a large amount of water constantly flowing into the excavation from fissures in the sandstone rock, to cover all the metal-work in the anchorages with several coats of tar-varnish, and to fill the pits with cement concrete. The links, however, should not be embedded in the concrete until they have received the full dead load of the structure; otherwise the extension of the links and the tightening of the joints due to the load might fracture the concrete. This point has an important bearing upon the efficiency of reinforced concrete.

If the anchorages had been left open, constant pumping would have been necessary; but as now built, they are permanently protected, and inspection is not necessary.

The Towers.

The construction of the towers, of which there are two on each side of the river, is indicated in Figs. 1, Plate 1, and is shown in detail in Figs. 5 and 6, Plate 2. The height from the top of the cylinders to the cable-saddle platform is 150 feet, to the centre of the cables 155 feet, to the lantern-platform 158 feet, and to the top of the lantern 180 feet. The towers are square in plan, with braced legs at each corner; the outside width at the cylinders is 35 feet, and at the top 9 feet, the outside profile of the legs being a curve of large radius. Each leg is built of twelve steel angles, 5 inches by 5 inches by ⅝ inch, arranged as shown in Figs. 6, Plate 2, and braced

between with steel angles, $3\frac{1}{2}$ inches by 2 inches by $\frac{5}{16}$ inch, the width of the legs at the base being 4 feet 10 inches, and at the top 2 feet 3 inches. The legs are braced horizontally with braced struts and diagonal flat tie-bars, the bottom struts consisting of four angle-bars, $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{3}{8}$ inch, placed 2 feet 3 inches apart vertically and 2 feet $1\frac{3}{4}$ inch horizontally, and the angles being braced each side with flat bars $2\frac{3}{4}$ inches by $\frac{3}{8}$ inch, as shown. The diagonal ties consist of two flat bars, $6\frac{1}{2}$ inches by $\frac{3}{4}$ inch at the bottom and $3\frac{1}{2}$ inches by $\frac{5}{8}$ inch at the top. The horizontal struts divide the towers into eight bays, the scantling of the struts and ties decreasing as the width of the tower decreases. Each pair of towers is braced together by a deep curved double-braced girder at a point just above the level of the top of the stiffening-girders, and by another at the top, level with the lantern-floor.

The top of the tower is plated, and is stiffened with cross girders and bearers to receive the bottom bed-plate of the cable-saddle rollers, so that the load may be transmitted equally to the four legs of the towers. The bottoms of the tower-legs are plated all round and securely bolted to the cylinders by eight bolts, 2 inches in diameter, which are built into the concrete filling the cylinders. The top of each tower is surmounted by an octagonal lantern, 7 feet 6 inches wide, and 8 feet 6 inches high to the eaves of the copper-covered curved roof; the framework of the lantern is of steel, fitted with timber panels and window-frames. Outside the lantern is a shallow balcony, with an ornamental railing 4 feet high; wire netting is fixed above to prevent people from getting outside. There is an iron spiral staircase in the centre of one of each pair of towers, from the ground-level to the top of the tower; and there is a gangway between the towers, on the top of the top bracing girder, and also one the whole length of the stiffening-girder.

The question whether the towers should be hinged or fixed at the bottom was considered when designing them; and although each plan has its advantages and disadvantages, it was considered that the fixed base would give the more satisfactory result as far as the stresses were concerned, and that, with the curved profile, it would undoubtedly have a more graceful appearance.

In calculating the strength of the various members of the tower, the loads taken into consideration were the vertical component of the cable-stresses at the towers, the weight of the towers and the wind-pressure. The wind-pressure on the stiffening-girders and the four towers was assumed to be transmitted to the leeward towers. The resultant forces acting on each member were then calculated, members in compression being treated as columns fixed at each end.

The maximum resultant pressure on the base of the two leeward cylinders is 5·77 tons per square foot; and the unit stress in the various members of the towers does not exceed 6½ tons per square inch in tension, that in the struts varying with the ratio of the length to the radius of gyration, and the ultimate strength of the material being taken as 28 tons per square inch.

The vertical component of the cable-stresses at the towers due to the loads and temperature and to the weight of the towers, can be ascertained with fair accuracy; but the wind-pressure presents problems which are not so easily solved.

Much has been written, and many experiments have been made, in connection with wind-pressure; but engineers are still in the dark on many points which have an important bearing upon the design of large structures. The extensive experiments of Sir Benjamin Baker at the Forth bridge, and those more recently carried out at the National Physical Laboratory[1] have added considerably to the knowledge of the subject, and the relation between the velocity and pressure has been fairly well established. Several interesting experiments were also made with a view to ascertain the relative pressure on two surfaces not in the same plane but in the same line of pressure; and although much valuable information was obtained, the experiments will require to be continued on a more extended scale before the deductions can be wholly accepted.

As showing the unreliability of the records of pressure and velocity made by the best instruments available at the time, the following records taken at Bidston Observatory during very high winds may be given:—

Date.	Velocity.	Pressure.	Direction of Wind.
	Miles per Hour.	Lbs. per Sq. Ft.	
27 December, 1868	92	80	W.
13 October, 1870	82	65	W.S.W.
9 March, 1871	79	90	W.
27 September, 1875	81	70	W.S.W.
23 November, 1877.	80	64	W.
28 December, 1879	59	38	S.W.

A Robinson anemometer was used for ascertaining the velocity, and an Osler anemometer for recording the pressure, and similar instruments have been used until recently at most of the well-known

[1] T. E. Stanton, "On the Resistance of Plane Surfaces in a Uniform Current of Air." Minutes of Proceedings Inst. C.E., vol. clvi, p. 78.

meteorological stations. These results are totally unreliable: in one instance, a velocity of 79 miles per hour has a corresponding pressure of 90 lbs. per square foot; while at another time a velocity of 80 miles per hour has a pressure of only 64 lbs. per square foot.

Several explanations have been suggested for these abnormally high recorded pressures, such as the impetus of the recording disk, or a large bird having been blown against it; but the facts point to one conclusion, namely, that the instruments used were quite unreliable for high pressures. At present, no reliable records exist of actual high pressures or velocities, on account of the many difficulties attendant on the observations, and the absence of reliable instruments for measuring and recording them accurately. Even supposing that, from the researches at the National Physical Laboratory or elsewhere, definite and reliable results are obtained as to pressures and velocities, and their relative values under various conditions, an important point in connection with the action of the wind on large structures will still remain unsolved.

The wind on a structure cannot be treated solely as a static pressure: it is quite possible for a structure to withstand a high wind-pressure, and yet be destroyed by a wind of considerably less pressure, for the following reasons:—Every structure as a whole, and each individual member of the structure, is tuned to a certain pitch under a given load. Wind-pressure is not an even pressure, but comes in gusts at varying intervals; if therefore a structure is set in vibration by a gust of wind, and the successive gusts of wind synchronize with the pitch of the structure, the vibration increases in intensity with each gust until disaster follows. This action is well illustrated when a man walks along a long single plank, and by the well-known plan of moving a large heavy box or other article by a succession of slight tilts, when it is too heavy to be moved by direct pressure.

An instance of this action was experienced during the erection of the Widnes and Runcorn bridge. The two main cables had been erected in position, and the clips for the stiffening-girder suspenders had been bolted to them. A gale of wind arose while the cables were thus freely suspended, and a longitudinal vibration was set up in them, the wave being 10 feet in height, and having a period of about 4 seconds. In the following week, before the cables had been secured by guys, another gale sprang up from exactly the same quarter, but of greater violence than the previous one, and the cables were unaffected.

Another instance of similar effects occurred when the Menai suspension-bridge was nearly destroyed in 1904; during a strong gale the suspended roadway was torn away from each pier, and the whole roadway, being clear of the masonry, was swinging laterally

to the extent of 4 or 5 feet. The bridge had withstood far heavier gales previously without sustaining any damage; but on this occasion the vibration of the roadway—which is very considerable during a gale, as the bridge is not stiffened—evidently synchronized with the gusts of wind and caused the rupture.

These results, which might possibly occur to any large structure, cannot be foretold from either researches or experiments; but, fortunately, it has been clearly established that a gale of wind acts seldom, if ever, with uniform pressure over a very large surface, and dangerous cumulative vibrations may be often thus counteracted: the danger, however, always exists.

In calculating the stresses in the towers, stiffening-girders, etc., a pressure of 56 lbs. per square foot over the whole of the visible front surface and of 40 lbs. on the corresponding back surface, was allowed for, but the pressure on the car was not included, for it would certainly not be running when such a gale was blowing. These pressures are in accordance with the requirements of the Board of Trade; but in the opinion of the Author they are excessive; for as the track of a tornado has never been known to exceed 300 feet in width, it is not at all probable that a pressure of 56 lbs. per square foot would act simultaneously over a width of 1,000 feet. Possibly, when investigations of the action of the wind upon structures, more scientific than those upon which the Board's regulations were founded, have been satisfactorily completed, more reasonable conditions will be enforced.

The Cables.

In designing the cables, it was necessary to take into consideration the facility of erection, the surface exposed to wind and weather, the attachment to the anchorages, and the connections with the stiffening-girder suspenders.

In existing cable suspension-bridges of large span, the form and construction of the cables vary considerably. In some bridges the required sectional area is divided among several small cables, on the ground that they are more easily constructed and erected; in others—as in the Brooklyn bridge and the new East River bridge at New York—the whole of the required area is contained in four cables, two over each longitudinal girder. Again, the construction of the cables varies; in some—notably recent French bridges—the cables are jointed at the top of the towers; in others, as in the New York and most other bridges, the cables are continuous from anchorage to anchorage. The cables in some are formed of concentric twisted

strands like an ordinary rope, while in others, *e.g.*, the two New York bridges, they consist of individual wires laid parallel throughout and bound together.

Taking the various existing methods of construction into consideration, it was thought that to adopt a number of cables, although it might facilitate erection, would have the objection of exposing a much larger surface to the action of the chemical-laden atmosphere of the district, and would multiply the various connections and attachments.

It was therefore decided to have one cable over each stiffening-girder, and to make it continuous from anchorage to anchorage. If a cable is constructed of a number of parallel wires bound together, although each wire may be laid with the same versed sine, there must be a probability of the wires getting displaced when being drawn together for binding; and it would appear to be almost impossible to ensure the wires being all parallel, in which case they would not be all strained alike. The supposed advantage of having all the wires laid parallel is that the cable will have a higher breaking-load than one consisting of a number of twisted strands. Experiments were made at the United States Arsenal at Watertown, Mass., in 1895, with four ropes made of parallel wires and four laid with twisted wires, each rope being constructed of thirty-seven wires of No. 8 gauge. The results[1] showed that the wires were much more uniformly strained in the twisted ropes than in those with the straight wires; the ultimate breaking-loads of the latter were slightly more than those of the twisted ropes, whose modulus of elasticity was 25,000,000 lbs. per square inch against 27,000,000 lbs. for the straight-wire rope. Several sample ropes for the Widnes and Runcorn bridge were tested at the Birkenhead Testing Works and at the Sheffield Testing Works; and the results agreed very closely with those arrived at in the Watertown experiments.

One undoubted advantage of the cable laid with straight wires lies in its weight; a twisted-wire rope 2 inches in diameter, with a lay of 24 inches, weighs 3·75 per cent. more than one with straight wires. The sectional area, however, of the twisted-wire rope, taken normal to the centre line of the rope, is correspondingly larger; and the question arises whether or not it is correct to take this increased area in calculating the effective sectional area. It would appear, then, that the only disadvantage of the twisted rope is its lower modulus of elasticity; but it has the distinct advantage of the certainty that all the wires are uniformly strained. As the

[1] Proceedings of the American Society of Civil Engineers, vol. xxii, p. 479.

lower modulus does not in any way affect the strength of the rope, but only increases slightly the extension and consequent deflection of the cable, it was decided to build the cables with strands of twisted wire.

In making the tests of the rope, it was found that the outside wires always broke first, which is no doubt due to their firmer grip in the sockets, those in the centre drawing slightly. This does not support the generally accepted idea that the central wire of a laid rope, being straight, is of no value because it will break first, on which assumption it has been the custom to omit the area of the central wire in calculating the effective area of bridge-cables. Even supposing the central wire did break first, the cables of a bridge are not supposed to be strained beyond their elastic limit, and up to that point the central wire has certainly been doing duty, and should be included in the sectional area of the cables.

Very few experiments carried out in England on the physical properties of steel wire are recorded, and none give the actual yield-point. This has been generally assumed to be the same as for ordinary steel bars—about 60 per cent. of the breaking-weight; but in the correspondence in American journals with reference to the safety of the Brooklyn bridge,[1] Mr. W. Hildenbrand, Chief Engineer for the cables of the new East River bridge, refers to a number of experiments which gave an elastic limit as high as 85 per cent. of the breaking-weight. In correspondence on the subject between the Author and Mr. Hildenbrand, the latter stated that when building the Cincinnati suspension-bridge, he engaged Mr. G. C. Henning, a well-known testing-expert, to find the exact limit of elasticity, and also the modulus of elasticity of the wire. Mr. Henning designed a special recording-apparatus, and after hundreds of experiments he found that the elastic limit was at least 85 per cent. of the ultimate strength, and not infrequently 90 to 95 per cent. This was the first discovery or record of this very important result, which affects considerably the question of the factor of safety to be adopted in designing cables.

The wires for the cables of the Widnes and Runcorn bridge are 0·162 inch in diameter, and were specified to have a breaking-load of 90 to 95 tons per square inch. Each wire was tested, and the results of about five thousand tests gave an average breaking-load of about 94 tons per square inch, every coil of wire which broke with less than 90 tons being rejected. Several experiments were made to ascertain the modulus of elasticity, the average working out at

[1] *Engineering News*, vol. xlvii, p. 53.

29,126,650 lbs. per square inch; but the yield-point was not ascertained, no reliable apparatus for recording it being available.

The cables, shown in section in Fig. 7, Plate 2, are formed of nineteen strands, each strand containing one hundred and twenty-seven wires, the central wire being straight, and the others laid in two concentric rings with a lay of 24 inches, the rings being laid alternately left- and right-handed. The wires are all laid at one angle, so that in a given length of cable the wires of each concentric ring are of the same length.

Several experiments were made with strands having the wires of the concentric rings all laid in the same direction, and others with the wires laid alternately in opposite directions: the results were strongly in favour of the latter.

The strands of the cables are laid parallel, and are bound together at intervals of about 18 feet 6 inches with strong cast-iron clips (Figs. 8, Plate 2), constructed to receive the forged steel bridles, from which the suspenders for the stiffening-girders are hung. The strands are also bound, at intervals of about 6 feet, with bands of soft steel wire, No. 6 S.W.G. During the construction of the strands, the interstices between the wires were filled with a bituminous compound, and when the cables were completed, they were wrapped with two layers of strong sailcloth saturated with the same compound.

The ends of the nineteen cable-strands are fitted into sockets as shown in Figs. 9, Plate 2: into the sockets fit steel screws 4 inches in diameter, with a forged end to receive the pin which passes through the ears of the socket. The other end of the screw passes through a steel cross-head, the nut being placed at the back, by means of which the length of the cable-strand could be adjusted. The nineteen strands are divided into three groups, of six, seven and six respectively, attached to three cross-heads, which are coupled to the anchor-links, the seven-strand cross-head being in the centre. At about 60 feet from the cross-head, the nineteen strands are gathered together and clipped with a bell-mouthed steel casting.

The cross-heads were originally designed to be of cast steel; but as the whole structure is dependent upon their strength, it was thought that, as there were certain risks from hidden flaws, it would be safer to make them of compressed forgings. These were accordingly made in an armour-press at the works of Messrs. William Beardmore and Co., and they are most interesting and, to a certain extent, novel pieces of work. Special moulds or dies were made, approaching the cross-heads in shape, between which the bloom was placed, and submitted to a hydraulic pressure of 3,500 tons. When the forgings—weighing about 14 cwt. each—were cold, they were cut

with a fret-saw into the required shape. The sockets for the cable-strand ends are also pressed forgings.

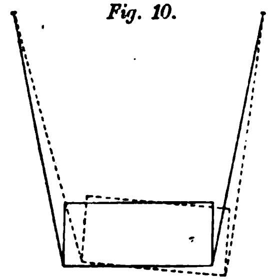
Before deciding upon the method of fixing the strands into the sockets, several experiments were made with different plans. The usual plan is to open out the wires of the strands, and drive small steel wedges into the interstices, the ends of the wires being then turned over and the whole run with white metal. This method gives very good results, and if it is properly carried out, the strand will break outside the socket. There is, however, an uncertainty about the wedging being properly done, and the process is a long one if it is done satisfactorily. A more satisfactory plan is to have a number of concentric tapered wedges, instead of the small needle wedges; the wires are opened out as before, and the wedges, about four to each ring, are driven in between each ring of wires; the ends of the wires are turned over, and the socket, having been previously heated, is run with white metal. By this method, designed by Mr. W. J. Glover, of the St. Helens Cable Company, the grip of each wire by the wedges is ensured, and there is no necessity to pick out the right thickness of wedge for the interstice as in the old plan, each wedge being in the same position in each socket. Several full-sized strands were tested with this connection, and in no case did the socket fail, the strand always breaking outside.

The cable-strands for the bridge were attached to the sockets in the foregoing manner; but the socket connections of the steel-rope suspenders of the stiffening-girders were constructed differently. These suspenders are ordinary twisted ropes, $1\frac{3}{4}$ inch in diameter, having a breaking-load of 104 tons; the socket connection was made by opening out the wires and bending each one down into a loop inside the socket; a steel annular wedge was then driven into the bottom of the socket, and the whole was run with white metal. With ropes of the ordinary construction this makes a perfect connection.

The main cables do not hang vertically when in position, but are cradled, the distance between the centres of the cables at the towers being 70 feet, and at the centre of the bridge 38 ft. About the value of cradling the cables there is a diversity of opinion; but there is not the slightest doubt that it stiffens the structure considerably—a fact which can be proved by the simple experiment of swinging an article with two strings, first with the strings parallel and next with them opened out wider at the top. It has been held that if the stiffening-girder with cradled cables is in its normal position, as shown by the full lines in *Fig. 10*, it will assume the position shown by the dotted lines when submitted to wind-pressure; the effect being to relieve

the load on the suspenders on one side, and put it on the other. In order to do this, however, it would be necessary to twist the stiffening-girder, because it is held down at the towers, and this action would also be resisted by the cross bracing of the girders. The cradling of the cables involves a little more care in setting out and in making the connections, but this extra trouble is more than made up for by the undoubted stiffness which cradling imparts to the structure.

Fig. 10.



The main cables have each a net sectional area of 50 square inches, and at a temperature of 60° F. the versed sine of the 1,000-foot span is 84 feet 4 inches. Under the maximum dead and live loads and wind-pressure, the cables are subjected to a stress of 21·24 tons per square inch, the average breaking-load of the wires being 94 tons and of the cable-strands 84·6 tons per square inch.[1] With a difference of temperature of 5° F. the cable would rise or fall about 1 inch if simply suspended alone; but when it is attached to a stiffening-girder with a hinged centre the movements are somewhat complex. A very interesting discussion on this matter took place in the United States,[2] when the safety of the East River bridge was questioned, after the accident to the suspenders in 1901.

In order to allow for the extension of the backstays due to the live load and to variation of temperature, the cables rest upon cast-iron saddles and rollers (Figs. 11, Plate 2). The angle of the backstays at the towers being sharper than that of the cables, it was necessary to clamp the cables to the saddles, the tendency of the resultant pressure being to force the saddles towards the centre of the bridge. The cable and saddle being thus united, the resultant pressure is of course vertical through the rollers to the towers. The rollers are not complete cylinders, but are of the form shown in Figs. 11, so that a large number of rollers can be used with a minimum length of saddle and roller bed-plates. The rollers are 43 inches long and 12 inches high, the maximum pressure on them being $1\frac{1}{4}$ ton per lineal inch of rollers. There are seventeen turned rollers under each saddle, coupled together by two flat bars and pins on each side, to keep them parallel and equidistant one from another.

[1] Assuming the strength of the strands per square inch to be 10 per cent. less than that of the individual wire.

[2] *Engineering News*, vol. xlvi, p. 250 *et seq.*, and vol. xlvii, p. 53.

The Stiffening-Girders.

In determining the form of the stiffening-girders, the object of which is to transmit the load practically equally to all the suspension-rods, and so to minimize the distortion of the cable, the question of hinge or no hinge at the centre was considered.

If the girder is continuous, without a hinge at the centre, the stresses are to a certain extent indeterminate, and the stresses due to temperature may be very high; further, as the temperature-stresses increase with the depth of the girder, shallow girders would be necessary to minimize them. If the girder is hinged at the centre, the stresses are determinate, and the temperature-stresses are practically eliminated; but as the effect of the rolling load when concentrated at the centre is to double the load on the suspenders, the stresses in the cable and girders are increased accordingly. With a properly proportioned girder, the extra sectional area required to meet this additional stress in a hinged girder is considerably less than that required to meet the temperature-stresses in a girder with no hinge. Diagrams of the maximum bending-moments and shearing-forces of the live load and the wind-pressure, as well as of the varying tension in the suspenders, are shown in Figs. 12, Plate 2.

Fig. 13, Plate 2, is an elevation of one-half of the stiffening girder and various details of the joints and connections are shown in Figs. 14–17, Plate 2; details of the hinged joint at the centre are illustrated in Figs. 18 and 19, Plate 3. The girders are 1,000 feet long between the centres of the towers, and project 55 feet at each end to enable the transporter-car to run into a dock between the towers, making a total length of 1,110 feet; they are 19 feet deep over the backs of the outside angles, and are 35 feet apart between centres.

As the live load in certain positions tends to raise the girders at the ends, it is necessary to hold them down; and at the same time, expansion and contraction from temperature-changes must be allowed for. This is effected by means of rockers, which are coupled to each of the girders near the centre of the tower and to bearings on girders fixed between the tower-cylinders. The girders beyond the rockers act as cantilevers; the rockers take the greater part of the weight of the car and trolley, the remainder being taken by a suspender directly over the rocker.

The construction of the stiffening-girders is of the ordinary plated trough section for the top and bottom booms, with vertical braced struts and diagonal tension-members, consisting of round bars. Details of the various sections are shown in Figs. 16 and 17, Plate 2.

In designing the girders the wind-pressure was an important factor, and an endeavour was made to present as little surface as possible in the various members: for this reason the booms are shallow, and the tension-members are round bars, varying in diameter according to the different shearing-stresses. With an assumed pressure of 56 lbs. per square foot on the windward face of the girder, and of 40 lbs. per square foot on the leeward girder, the effective pressure on the girders amounts to 140 tons. The two girders are braced horizontally between the top and bottom booms, forming a double horizontal girder, to withstand this wind-pressure. The pressure is transmitted to the towers by horizontal rubbing girders with slightly curved faces, the adjustable rubbers being of manganese steel.

The bottom booms of the girders are constructed to carry the rail upon which the car-trolley runs on the outside bottom angle, as shown in Fig. 17, Plate 2. The objection to carrying the weight of the car-trolley on the outside rail is that the load is not transmitted to the centre of the girders; but if a special carrier for the rail had been attached to the underside of the girder there would have been a considerable increase in the surface exposed to wind-pressure. The steel rails, in addition to being bolted to the bottom angle-bar, are fastened at short intervals by steel castings to the outside vertical member of the boom.

The suspenders from the cables are attached to the inside of the bottom boom by means of a pin-connection (Figs. 16 and 17, Plate 2), two vertical diaphragms being riveted to the bottom plates, through which the pin passes; the diagonal tension-rods of the girder are also coupled to the same pin.

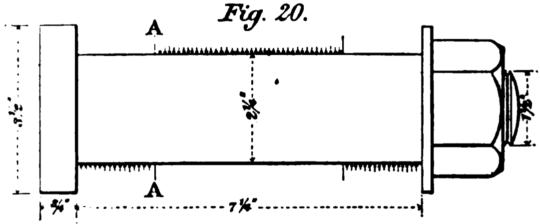
All the booms have rigid riveted joints, and the vertical struts, consisting of four angle-bars, varying from 3 inches by 3 inches by $\frac{3}{8}$ inch to 3 inches by 3 inches by $\frac{1}{2}$ inch, braced with flat bars $2\frac{1}{2}$ inches by $\frac{3}{8}$ inch, are also riveted to the booms, the diagonal bracing-bars having only pin-connections for convenience of erection.

In designing the pins for the different parts of the structure the bending-moments were ascertained, and the pins were so proportioned that the unit stress on the extreme fibre did not exceed $6\frac{1}{2}$ tons per square inch. If the pins had been designed to meet the shear stress only they would have been too small in diameter; and as the pins in bridgework are not usually tight-fitting, they would have bent under the action of the load, causing a yielding of the structure, which might have developed other stresses which it was not calculated to withstand. This is illustrated by the following example. Suppose a pin is required for a jaw-coupling as shown in *Fig. 20* upon the following assumptions:—

Tensile strength of material	. . .	30 tons per square inch.
Shear ,, ,, ,,	. . .	25 ,, ,, ,, ,,
Factor of safety	. . . .	4·6.
Total load on coupling	. . . .	162·6 tons.

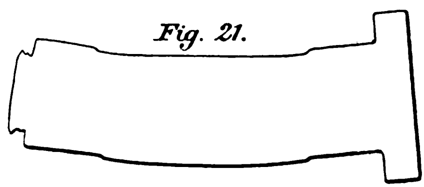
The bending-moment on a pin at line AA = 7·26 inch-tons = M. Then from the formula $M = \frac{fI}{y}$, where I = moment of inertia, and y = half diameter of pin; and taking $f = 6\frac{1}{2}$ tons, $2y = 2\cdot24$. Thus the required diameter of the pin to comply with the conditions of the bending-moment would be $2\frac{1}{4}$ inches.

Fig. 20.



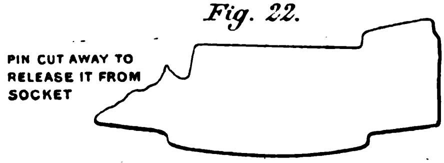
NO LOAD

Fig. 21.



LOAD 133·51 TONS

Fig. 22.



LOAD 162·6 TONS

SCALE 3 INCHES = 1 FOOT

If the pin is now assumed to be of this diameter, the average shearing-stress would be only 2·08 tons per square inch; and if the pin were designed for shear, the diameter would be considerably less, working out at $1\frac{7}{16}$ inch with a unit shear-stress of 5·12 tons per square inch.

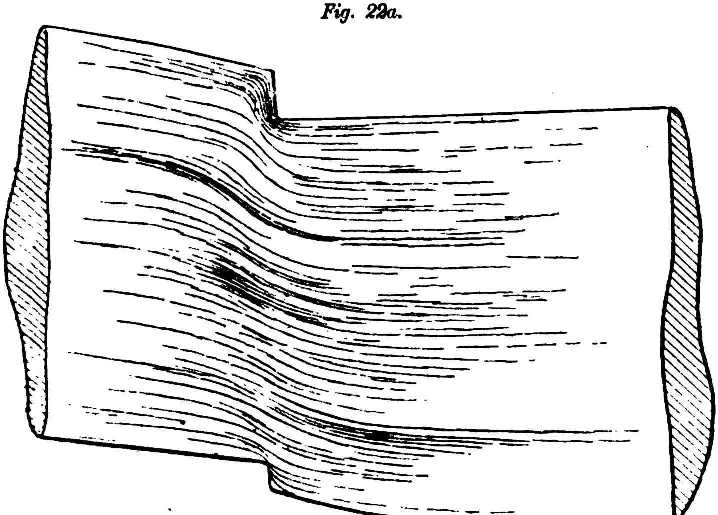
If the bending-moment is ascertained as before for this $1\frac{7}{16}$-inch pin, the extreme fibre-stress works out to 24·91 tons per square

inch. Now if this pin were loaded, it would take a permanent set when the extreme fibre-stress exceeded the elastic limit of the material, which may be assumed to be 14 tons per square inch. The pin would therefore fail with a load of 9·33 tons, which produces this extreme fibre-stress of 14 tons; that is to say, it would fail with a load 7·3 tons less than the load which it was designed to carry with a factor of safety of 4·88; which is about 44 per cent. of the load it was designed to carry safely for shear stress only.

A number of experiments were made with steel pins 2 inches and 2¼ inches diameter, which were tested almost to destruction when held as shown in *Fig. 20*.

The pin in *Fig. 21* was submitted to a load of 133·5 tons, and

Fig. 22a.



FULL SIZE LONGITUDINAL SECTION OF PIN SUBMITTED TO A LOAD OF 162·6 TONS.

was bent as shown, the shearing action being very marked. The pin in *Fig. 22* was submitted to a load of 162·5 tons with the result shown. *Fig. 22a* is a full-sized longitudinal section of the pin shown in *Fig. 22*. The average shearing-stress in the pin shown in *Fig. 21* was 16·7 tons per square inch; of that shown in *Fig. 22*, 20·4 tons per square inch. It is evident that the pin would ultimately fail by shearing, the tested pin in *Fig. 22a* showing the interesting flow of metal. This pin, after being tested, was sawn in halves longitudinally; the surface was then carefully prepared and etched to bring out the molecular structure or "grain" of the metal and

a photograph of it was taken, from which *Fig. 22a* has been prepared. For this work the Author is indebted to Mr. Bertram Blount, Assoc. Inst. C.E.

The following Table shows the results of tests on four pins under different loads :—

Diameter of Pin.	Average Shearing-Stress.	Result of Test.
Inches.	Tons per Square Inch.	
2	12·2	Pin bent, shear beginning to show.
2	13·35	Pin bent further and shear marked distinctly.
$2\frac{1}{4}$	16·7	Pin bent considerably, shear marked $\frac{1}{16}$ inch deep.
$2\frac{1}{4}$	20·4	Badly bent, shear mark $\frac{1}{4}$ inch deep.

The bearing unit-stress in all these pins was well within the ordinary factor of safety.

The extreme fibre-stress in the foregoing experiments has not been calculated, because the formula $M = f\,I/y$ is not applicable when the material is loaded beyond its elastic limit.

From the above calculations and experiments it is clear that, although a pin would fail by shearing if tested to destruction, it would be injured by bending if designed to withstand shear only; and in designing pin-connections, the bending-moment should therefore be ascertained and the pins be proportioned accordingly. The experiments also showed the necessity of having strong sides to the jaw-connections; otherwise the bending action of the pin under the load forces the jaws apart, and so increases the effective length of the pin as a beam. In several of the experiments, pins of the same diameter and same length between the bearings failed with totally different loads, from the above-mentioned causes.

The suspenders from the cables are attached to the stiffening-girders by socket-ends fitting into a cross-head, through which run two long adjusting-screws passing round a steel forging, which is coupled to the pin passing through the tension-bars in the bottom boom. Details of these connections are shown in Figs. 8, Plate 2.

In order to minimize the tendency for the girders to move slightly longitudinally under the action of the load, raking guys consisting of steel bars $3\frac{1}{4}$ inches in diameter are attached to the hinge-pins at the centre of the girder, and to the cables.

The Rockers.

The rockers supporting the girders at the towers are shown in Figs. 23, Plate 3; they are 69 feet $9\frac{1}{4}$ inches long between the

centres of the pins, and consist of four steel angle-bars 5 inches by 5 inches by $\frac{5}{8}$ inch, spaced 2 feet 6 inches over angles at the centre, and tapering to 12 inches at the ends. Strong steel castings are riveted to each end, and are bored out to receive the bearings, 7 inches in diameter, fixed to the sides of the stiffening-girders at the top (as shown in Fig. 24, Plate 3) and to the tower-cylinder bracing girders at the bottom. The angle-bars are braced on all sides with angles $3\frac{1}{2}$ inches by $1\frac{3}{4}$ inch by $\frac{3}{8}$ inch, and with flat bars 3 inches by $\frac{3}{8}$ inch.

Trolley and Transporter-Car.

The trolley, from which is suspended the transporter-car, is shown in Figs. 25 and 26, Plate 3. It consists of an articulated frame 77 feet long over all, in five sections coupled together by links and pins. To this frame are fixed the bearings upon which the running-wheels revolve. There are fourteen wheels on each side, 18 inches in diameter, running on fixed bearings $3\frac{1}{2}$ inches in diameter and 6 inches long, in addition to the four driving-wheels. The trolley frame is 4 feet 10 inches deep, and is built of channel-bars 7 inches by 3 inches and weighing 17 lbs. per foot, with flat-bar and channel bearings, each frame being braced horizontally.

The second and fourth sections of the trolley carry the driving-gear, and each is equipped with an enclosed steel-clad series-wound Mather-Platt electric motor. Each motor drives a pair of wheels 20 inches in diameter, by means of a 3-inch shaft coupled to each end of the motor armature-shaft, and through machine-cut steel gear-wheels 4 inches broad. The output of each motor is 36 B.HP. They are together capable of giving a tractive force of 8,000 lbs., and can be worked up to 10,000 lbs. for short periods. Thus, assuming a test-load of 120 tons, a tractive force of at least 70 lbs. per ton is available, which is ample for acceleration and for maintaining speed against wind-pressure. The driving-shafts are in sections, fitted with adjustable clutch couplings to allow for any faulty alignment due to the springing of the trolley; four of the clutches are fitted with levers for throwing the motors in and out of gear. Hand-gear, for driving the trolley in case of a failure in the electric supply, is fitted to each of the motor-gears. The trolley has four magnetic brakes, shown in Fig. 27, Plate 3, which are arranged to act upon the track-rail and wheels simultaneously. These brakes consist of a horse-shoe electro-magnet, suspended by spiral springs from the trolley framing, the two poles hanging downwards directly over the track-rail. The current generated by the

motors when connected as generators, and therefore as brakes, is passed round the coil of the magnet, which is drawn down and magnetically grips the rails, and, using the friction on the rail as an abutment, operates a brake-block on one wheel. A triple braking effect is thus provided, without any tendency to lift the car.

In addition to the four electric brakes, there are four hand-brakes (Fig. 28, Plate 3), so arranged that equal pressure is applied to a pair of wheels simultaneously. All the machinery of the trolley is under the control of the operator in the cabin at the top of the car.

The car is 55 feet long and 24 feet wide on the deck, and is constructed as shown in Figs. 29, Plate 3. The underframe consists of sixteen light trusses 2 feet 2 inches deep, built of top and bottom angles, braced with flat bars and angles, upon which is laid the deck of 4-inch creosoted planks, close-jointed longitudinally with iron tongues, and sheathed with greenheart planks 2 inches thick, protected in the wheel-track by flat bars spiked to the sheathing. Light steel vertical frames are attached to the ends of the deck trusses, which act as distance struts to which the car-suspenders are attached, and also carry the deck upon which the operator's cabin is fixed. The suspenders are steel ropes 3 inches in circumference having a breaking-load of 32 tons each; they are attached to the trolley and to the car by means of socket-joints fitted to adjustable bridles. The suspenders are placed diagonally, both longitudinally and transversely, so as to stiffen the car against wind-pressure. A covered shelter, 22 feet by 8 feet, with glazed sides and sliding doors, is fitted on one side of the car. A strong U-shaped vulcanized india-rubber buffer is fitted to each end of the car, in a recess the full width of the car.

Electric Machinery.

The generating-station for supplying current for the motors and for illuminating the structure was designed in consultation with Dr. Edward Hopkinson, M. Inst. C.E.

The station, which is shown in section in Figs. 30, Plate 3, is built within the base of the Widnes east tower, and has three floors; the ground-floor containing the gas-engines, dynamos, etc., the first floor containing the battery of accumulators, and the top floor carrying the water-tank. The bottom floor is constructed of heavy web-plate girders to carry the heavy machinery, and is secured to the tower-cylinders and covered with steel joists embedded in concrete; the first floor is of timber covered with sheet lead. The main framing of the building consists of strong cast-iron columns supporting

rolled joists and other sections, and is filled with timber panels, large window-frames, and a sliding door on the ground-floor.

There are two gas-engines (one being spare), with cylinders 16 inches in diameter and 21 inches stroke, each capable of developing 75 B.HP. when running at 210 revolutions per minute with an easy working-load. Each engine is fitted with a fly-wheel weighing 109 cwt., has outside end bearings, and is coupled direct to the dynamos; it has a balanced crank-shaft, high-speed sensitive governor, double tube-ignition, and is fitted with Mohler bearings. A wrought-iron silencer is fitted to the end of each exhaust-pipe. The main gas-supply pipe is 4 inches in diameter, with branches 3 inches in diameter to each engine. The water-tank on the top floor has a capacity of 3,000 gallons. The calorific value of the Widnes gas is 660 to 670 B.Th.U. per cubic foot, and it is supplied at the low price of 1*s*. 1*d*. per thousand cubic feet.

The generators consist of two direct-driven four-pole shunt-wound dynamos (one being spare), each of 48 kilowatts capacity, capable of giving a continuous current of 90 amperes at 525 volts at 210 revolutions per minute, and, for a short time, an overload of 106 amperes at 525 volts.

The accumulators, by the Chloride Electrical Storage Co., consist of 245 cells having a capacity of 90 ampere-hours when discharged at 90 amperes, and of 180 ampere-hours when discharged at 20 amperes. They can also be discharged at 180 amperes for short periods.

The booster generator runs at about 760 revolutions per minute, and automatically charges or discharges the storage-battery; it keeps the main generator-current approximately constant, while the line-current varies according to the load on the motors. It is so constructed that, with no line-current, it raises the voltage across the cells from 525 to 585 volts, thus charging the cells. As the line-current increases, the charging-current decreases, until, when the load on the line is normal, the booster gives no voltage and there is no current flowing in or out of the cells if fully charged; if the line-current exceeds this value, the booster assists the battery to discharge. For giving a topping charge to the battery, an auxiliary resistance is employed to raise the voltage of the booster.

The main switchboard in the engine-room consists of three panels.

Panel No. 1 controls both generators.
,, No. 2 ,, the booster generator.
,, No. 3 ,, the trolley-motor feeders, booster, motor and the lighting-circuit.

The generator-panel is fitted with two ammeters, two double-pole main switches, two double-pole main fuses, two shunt regulating

switches and resistances. The accumulator- and booster-panel is fitted with ammeters, ampere-hour meter, switches and fuses and motor starting-switch, to work the battery of cells and booster in conjunction with one generator. The feeder- and lighting-panel is fitted with one maximum automatic circuit-breaker, one voltmeter, two lightning-arresters and kicking coils, one watt-hour meter to record the total consumption of energy in the station, two double-pole lighting-switches and single-pole fuses, one station-voltmeter, one leakage-detector, one two-way switch and resistance connected to earth.

As already mentioned, the trolley is fitted with two steel-clad series-wound motors of 36 B.HP. The car is fitted with a series-parallel controller placed in the operator's cabin, with handle so arranged that the controller is locked when the handle is removed. The controller is fitted with a reversing-switch and an arrangement for cutting out either motor if disabled; also with resistances for starting the motors. The controller-handle can be moved through the "off" position so as to throw both motors in parallel, acting as generators and operating the magnetic brake by the current generated. An emergency-switch combined with an automatic circuit-breaker, with ammeter and kicking coils, are fixed in the operator's cabin.

The collecting-gear for taking the current from the overhead conductor consists of a spring slipper in contact with the overhead positive and negative conductors, which consist of steel T-bars, 3 inches by 3 inches by $\frac{3}{8}$ inch, fixed to insulators attached to the cross bracing of the stiffening-girders. The T-bars are in long lengths, with flexible copper bonds at the joints, fished with steel plates.

The feeders between the switchboard in the engine-room and the overhead feeders are of lead-covered insulated cable 19/13 S.W.G.

In each of the lanterns at the top of the four towers is fixed a cluster of 5-ampere arc-lamps, five lamps being in series. The car, trolley, approaches, toll-houses, generating-station and waiting-rooms are fitted with about one hundred 32-candle-power glow-lamps, plugs being fixed at intervals along the bridge for coupling to pilot lamps.

Methods of Erection.

Tower-Cylinders.—As the rock on the Widnes side of the river at the site of the towers was bare at low water, there was no difficulty in fixing the cast-iron cylinders for the tower-foundations; the segments were lifted from the river-wall to the foreshore below, and, after being rolled to the site, were picked up with an ordinary hand-crane and placed in position. The bottom segments were bolted to the rock, and the cylinders were filled with cement concrete in the

proportion of 4 to 1 in the bottom length next the rock, and 7 to 1 for the remaining length, except the top length upon which the tower-leg rested, where the proportion was 4 to 1. The tower holding-down bolts and washer-plates were built in position as the work proceeded, a space being left around each to allow of a little play for the bolts when the superstructure was being fixed in position, the holes being subsequently run with cement grout.

The foundation-cylinders on the Runcorn side, being in the Manchester Ship-Canal, were sunk by the pneumatic process. A timber-piled staging was constructed from the shore to the site of the cylinders, upon which was laid a tramway, and upon the staging around the cylinders was fixed a 10-ton steam travelling crane. Each of the cylinders was temporarily kept in position by a timber cradling attached to four piles; at starting, seven lengths were placed in position and bolted together, the turned faces of the joints being covered with red-lead paint, which was sufficient to make them perfectly air-tight. The air-lock—which was of an old pattern—was then bolted to the top lengths, and about 40 tons of steel rails were stacked on the top, leaving a space at the centre to get to the air-lock trap-door. Towards the end of the day, the wedges between the cylinders and the cradling were released, and the air-pressure reduced, when the cylinders at once commenced to sink. When the stack of rails had sunk to the level of the staging, the rails were removed to another cylinder. Three more lengths were then added, and a stack of about 70 tons of rails was placed on the top, the operation being repeated until the rock was reached. There were two men inside at the bottom, filling the buckets, which had a capacity of 1⅓ cubic foot each; one man on a platform about 6 feet below the cylinder-covers passed the buckets through the air-lock, and made the necessary signals to the man outside; and five or six men outside received and emptied the buckets into the wagons alongside. When the rock was reached two more men worked at the bottom. The largest number of buckets passed through in an hour was thirty-five. The average working air-pressure was 16 lbs. per square inch, but when the water was being blown out of the cylinders before starting, the pressure was about 20 lbs. per square inch. There were two air-locks at work, the weights being transferred from one cylinder to the other. The concrete was passed through the air-locks until a depth of about 12 feet had been put in; the air-lock was then removed, and the remainder of the concrete was tipped in the open.

Towers.—For the erection of the towers, a triangular steel-framed staging 120 feet high was erected between the two groups of four cylinders, and upon this was fixed a 5-ton steam derrick-crane

capable of lifting to the highest point of the towers. The various parts were thus lifted into position, and as they were fixed men followed up with pneumatic riveters. When the towers were completed, the saddles, rollers and bed-plates were lifted and bolted in position.

Cables.—Each of the nineteen strands forming a cable was placed in a position separately in the following manner:—

A temporary frame was erected on the top of each tower, over which passed a "blondin" rope $1\frac{5}{8}$ inch in diameter, secured to the erection eye-bolts built in the anchorages. Upon these "blondin" ropes ran carriers, actuated by a hauling-rope worked by a strong steam-winch. The strand to be erected was coiled round a drum on the Widnes side; the free end, having been fitted to the socket before leaving the works, was attached to a carrier, and hauling began. At about every 50 feet another carrier was attached, and so on, until the socketed end of the strand was hauled to the anchorage on the Runcorn side. The socket was then attached to the adjusting-screw, and the strand was pulled and temporarily held as nearly as possible in its permanent position; it was then cut off to the correct length by means of powerful hydraulic shears, and, after being fitted to the socket, it was attached to the adjusting-screw at the Widnes anchorage.

Before fixing the cable-strands it was necessary to ascertain the versed sine with which they must be laid, so that the strand would be in its correct position when carrying its share of the load. A length of cable-strand 75 feet long was tested, and the elongation was carefully measured. From these observations it was calculated that to allow for the stretching of the cables and the straightening of the backstays it would be necessary to lay the cables with a versed sine of 76·3 feet instead of one of 84·09 feet. The subsequent erection of the structure proved this amount to be practically correct. Each of the nineteen strands was laid in a similar manner, and after being carefully adjusted they were temporarily clamped together until all had been laid, when the permanent clips were attached.

The undulation of the cable from the wind-pressure during a gale, already referred to, had a curious effect upon the clips. The action of the various strands sliding over one another caused the cast-iron clips to creep; and it was found, after the gale had subsided, that the top clip had travelled down the cable about 3 feet 6 inches, and the other clips in proportion. The clips were originally put on metal to metal, but after this experience they were removed, and a sheet of stout sailcloth, soaked in melted glue and covered with emery powder, was placed under the top clip; and, as an additional precaution, an

auxiliary cable was fixed from clip to clip and on to the saddle, to keep them in their correct positions. It is not possible for such an undulation to occur now that the cables are secured to the girders; but the slightest undulation of the cable will bring about the above creeping action to a certain extent, especially when the clips are carrying their load, and it is therefore advisable to adopt some precautionary measures to counteract it.

Stiffening-girders.—Before beginning the erection of the structure the contractors constructed a large model of the bridge to a scale of 10 feet to the inch, and to a weight-scale of half an ounce to the ton. From this model, fairly accurate indications could be obtained of the effects of the different loads on the cables and girders during construction. It was decided to build out the girders from each end. The cantilever ends were first built, and were secured by temporary guys and timber struts until the cable-suspenders were able to support the portion of the girder built beyond the rockers. The various portions of the girders were lifted by the crane on the staging, and placed on a carriage running on temporary rails laid on top of the girders as they were constructed. The pieces were then carried out and placed in position, no scaffolding or staging being used beyond a few planks at the end, as the work proceeded. The greatest difficulty the erectors had to contend with was the drawing down of the suspender-sockets to their position, for as the weight on the cables increased, the cables straightened and drew away the sockets beyond the length allowed in the adjustable screw attachment. This difficulty was overcome by using auxiliary screw attachments.

In the first stage of the erection, part of the load was carried by the rockers and part by the suspenders, and it was essential to know whether either were being over-loaded, or were taking more than their share of the load. This was ascertained approximately, and nearly enough for purposes of erection, by taking the angles of the cable from suspender to suspender and then the deflection of the cables at the centre of the bridge. Knowing the weights and the distances, a stress-diagram was easily constructed, and the tension in the suspenders was measured. As the construction of the girders proceeded, the dip of the cable at the centre became flatter and flatter, until it was not possible to measure it; and then this stress-diagram could not be constructed. A special apparatus was therefore made, which was attached to the booms of the girder opposite any suspender: it consisted of a multiplying lever coupled to a suspender, through which a force was applied to bend the suspender. The force applied was measured by a spring-balance, and the

deflection of the suspender was indicated by the end of the multiplying lever which travelled on a graduated index. From such data it was possible to ascertain fairly accurately the tension in all of the suspenders. When the girders were completed, the suspenders were all adjusted, the car-trolley was erected on the rails, and the car was suspended from it.

The bridge was formally opened for public traffic by Sir John T. Brunner, Bart., M.P., on the 29th May, 1905. During that day passengers were taken across the river at the rate of more than 2,000 per hour, in addition to a large number of vehicles, and upon several trips there were about 600 passengers and two vehicles on board. When working in parallel, with a load of about 10 tons, the time occupied in crossing the 1,000 feet was $1\frac{3}{4}$ minute; but as the traffic does not demand this quick running, the time now occupied is between 3 and 4 minutes, the power required varying from 25 B.HP. upwards according to the pressure and direction of the wind.

Cost of the Structure.

The total cost of the structure was about £133,000, including parliamentary expenses.

Abstracts of the specification for the various materials and of the contract schedule prices are given in Appendixes I and II.

The Engineers for the work were the Author and Mr. John T. Wood, M. Inst. C.E., the Resident Engineer being Mr. L. H. Chase, M. Inst. C.E. The contractors for the excavation, masonry, etc., were Messrs. W. Thornton and Sons, of Liverpool; the contractors for the superstructure were Messrs. Arrol's Bridge and Roof Company, Glasgow. The cables were made and erected by the St. Helens Cable Co., and the whole of the electric machinery for power and lighting was constructed and erected by Messrs. Mather and Platt, of Manchester.

The Paper is accompanied by nineteen sheets of drawings, one tracing, and two photographs, from which Plates 1–3 and the Figures in the text have been prepared.

[Appendixes.

APPENDIXES.

APPENDIX I.

Abstract of the Specification for the Materials.

Cement.—The cement to be the best Portland cement, by an approved maker, and to be so ground that 90 per cent. by weight will pass through a sieve with 2,500 meshes per square inch. It must weigh not less than 108 lbs. nor more than 120 lbs. per striked Imperial bushel; the bushel to be filled from a hopper, the spout of which is 18 inches above the mouth of the bushel. Test blocks, 1 inch square, mixed with water alone, and immersed for 7 days in water, must withstand a tensile stress of 400 lbs. per square inch before fracture. No signs of blowing, cracks, or heating are to appear in the sample blocks.

Concrete.—All concrete used in any portion of the work is to be composed of shingle, clean sharp river-sand, and Portland cement, in the following proportion by measure :—

For the backing of the retaining-walls	8 to 1.
For the filling of the foundation-cylinders	7 to 1.
For the filling of the anchorages	7 to 1.
For bedding the outsides of the Widnes cylinders and columns .	4 to 1.
For the foundation of the roadways	9 to 1.
The proportion of shingle to sand in all cases to be about .	5 to 4.

The materials are to be accurately measured in specially-constructed gauging-boxes, and then thoroughly mixed when in a dry condition and completely turned over twice; after which the proper quantity of water is to be added, and the whole is to be completely turned over three times before being dropped into the work. All the shingle and sand must be perfectly clean, and free from any mould or dirt, and if the engineers consider it to be necessary, they can order the same to be washed to their satisfaction. The concrete is to be well rammed in such places as the filling of the anchorage-pits, the cylinders, and any other places where they may consider it to be necessary.

Cast Iron.—All the cast iron to be of the best and toughest description, and the castings must be perfectly sound, clean, and free from air- or sand-holes, or other defects.

The mixture of the metal will be left to the judgment of the contractor, but it is to be of such a quality that a bar of it cast 3 feet 6 inches long by 2 inches by 1 inch when placed on edge on bearings 3 feet apart will not break with a less weight than 28 cwt. applied at the centre, nor deflect less than $\frac{5}{16}$ of an inch.

Plates, Bars, Angles, etc.—All the steel for the stiffening-girders, towers, approach-girders, and car is to be open-hearth acid steel, but the secondary

bracing-channels and bars may be of basic steel. The chemical constituents of the steel are left to the judgment of the maker, but all the steel must withstand the physical tests specified hereafter.

All the plates, bars, angles or other sections of steel are to have a clear, smooth skin, free from all scales, blisters, cracks, flaws, or other defects, and must be rolled to the full sections and weight per foot either shown on the drawings or specified. They must be of such a quality as to stand the tests specified hereafter, and the brand and maker's name is to be submitted to the engineers and receive their approval before the work is proceeded with.

The whole of the plates, bars, angles and other sections required for the principal members of the superstructures, which are made from acid steel, are to be able to withstand a stress of not less than 28 tons nor more than 32 tons per square inch before fracture; and the average of the tests must not be less than 30 tons per square inch of the original sectional area.[1] The elastic limit of the material must be not less than 16 tons per square inch, and it must stretch not less than 22 per cent. in a length of 8 inches before fracture. The channel bars and other sections required for the secondary bracing of the members may be of basic steel, and must withstand a tensile stress of not less than 27 tons nor more than 30 tons per square inch of the original section, and must stretch 20 per cent. in a length of 8 inches before fracture. The steel for the rivets must be able to withstand a tensile stress of not less than 24 tons nor more than 27 tons per square inch of the original section, and must stretch not less than 26 per cent. in a length of 8 inches before fracture.

Strips of the steel must be able to withstand being bent when cold through 180 degrees over a round bar whose diameter is equal to the thickness of the test-piece, without showing any signs of fracture; a rivet must be able to withstand being bent double upon itself when cold without signs of fracture or splitting.

Steel Castings.—All steel castings are to be cast true to pattern, and to be free from all blemishes, shrinkage, cracks, or other flaws, and all bearing-surfaces must be solid and free from any porosity. Large castings are to be suspended and hammered all over, and none of the above flaws and defects are to appear after such treatment. When possible, by casting a strip on any casting, or otherwise, a test-piece is to be turned having a gauged length of 2 inches and a diameter of ½ inch, which must withstand a tensile stress of not less than 30 tons nor more than 32 tons per square inch of original section; it must stretch not less than 18 per cent. in a length of 2 inches, and must have an elastic limit of not less than 15 tons per square inch.

When it is impracticable to obtain test-pieces, borings must be submitted to chemical analysis, and they must not contain more or less than 0·40 per cent. of carbon, and not more than 0·05 per cent. each of phosphorus and of sulphur.

Steel Forgings.—All forgings are to be free from cracks, flaws, seams or any other injurious imperfections. The heads of all bolts are to be forged in one with the bolt, and are to be of the same thickness as the diameter of the bolt, and neatly finished with six sides. The ends of the eye-bars are to be forged out of the solid, by previously "upsetting," and upon no consideration will welding be allowed. Five full-sized tests will be made of the eye-bars forming the diagonal bracing of the stiffening-girders, and they will be required to withstand a tensile stress of not less than 28 tons per square inch, and to stretch 14 per cent. in a length of 15 feet, and the bar must break in the body and not in the head;

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[1] Several of the manufacturers objected to the "average" clause, and refused to guarantee it, but in every case the specified amount was exceeded.

provided that not more than two of the bars break through the head and that these bars have withstood the tensile stress and given the elongation specified, they will not be rejected. Upon no consideration must any forgings be worked when at a blue heat.

Steel Wire.—The chemical composition of the steel required for manufacturing the wire for the cables is left to the judgment of the manufacturer, but it must be capable of withstanding a tensile stress of not less than 90 tons nor more than 95 tons per square inch before fracture, and of stretching 5 per cent. in a length of 8 inches and 2½ per cent. in a length of 5 feet; it must have a modulus of elasticity of 13,393 tons per square inch.

The wire for the cables must be drawn to a diameter of 0·162 inch, and the wires must be in one length from anchorage to anchorage, no splicing being permitted. The wire forming the core of each rope is to be slightly larger in diameter, and to have a modulus of elasticity of about 10,000 tons per square inch. The wires forming the cable ropes are to be able to be bent four times round a bar ½ inch in diameter when cold, without showing any signs of failing.

The wires for the main suspenders, and for the car-suspenders, are to be made of the same class and quality of steel, but the wires are to be drawn to a smaller gauge, and the ropes are to be of the ordinary construction and of the sizes and strengths already specified. The main and car-suspenders must be galvanized and the other ropes bright; the galvanized ropes to be subsequently painted as directed.

Summary of Tests.

——	Maximum Stress.	Minimum Stress.	Average Stress.	Limit of Elasticity.	Extension Per Cent.
	Tons Per Sq. In.	Tons Per Sq. In.	Tons Per Sq. In.	Tons Per Sq. In.	In 8 Ins.
Plates, bars, angles or other sections in principal members—(acid steel)	32	28	30	16	22
Secondary bracing—(basic steel)	30	27	..	15	20
Steel for rivets	27	24	..	13½	26
Steel wire in cables, suspenders, etc.	95	90	..	..	5
					In 2 Ins.
Steel castings	32	30	..	15	18

APPENDIX II.

Abstract of Contract Prices.

		£	s.	d.
Excavation and wheeling for retaining-walls	per cubic yard	0	2	6
Filling in between retaining-walls	„ „ „	0	3	0
Pitch-pine piles 12 inches by 12 inches about 15 feet long	„ „ foot	0	1	9
Pointing, shoeing and cutting piles	per pile	0	17	6
Driving 84 piles to a depth of 13 feet	„ „	1	0	0
Portland-cement concrete in retaining-walls, 8 to 1	per cubic yard	0	16	0
Portland-cement concrete in cylinders, 7 to 1	„ „ „	2	4	0
Stone face of walls, average 12 inches thick	per superficial foot	0	3	6
Granite macadam roadway	per square yard	0	4	6
Wood-block roadway	„ „ „	0	10	0
Granolithic pavement	„ „ „	0	7	0
Excavation in rock for Widnes cylinders	per cubic yard	1	2	0
Sinking 8 cylinders to rock on Runcorn side to a depth of 27 feet	each	50	0	0
Cast-iron cylinders erected	per ton	12	2	6
Cast-iron columns 14 inches in diameter for approach-girders	„ „	12	2	6
Steel girders for approaches	„ „	19	17	6
„ „ „ stiffening-girders	„ „	20	7	6
Towers	„ „	19	15	0
Corrugated flooring plate for subway	„ „	16	2	6
Cast-iron saddle-plates	„ „	17	17	6
„ „ rollers	„ „	19	0	0
Cast-steel bearings for rockers	„ „	39	5	0
Small steel castings	„ „	56	0	0
Manganese steel rubbers for stiffening-girder	„ „	61	15	0
Forged steel cross-heads for anchorages	„ „	60	0	0
Cables and suspenders fixed complete	„ „	45	0	0
Steel sockets for cable-strands and suspenders	„ „	67	10	0

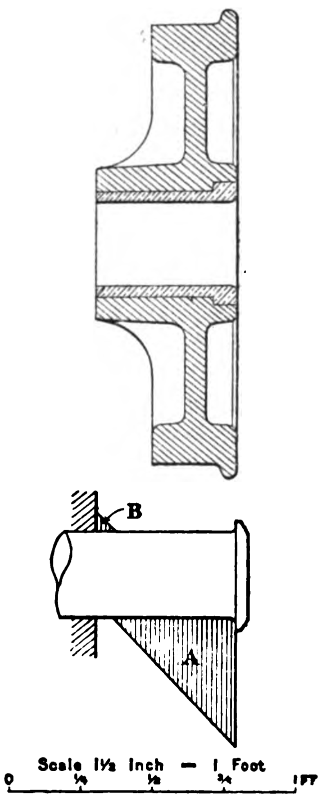
Discussion.

The PRESIDENT, in moving a vote of thanks to the Author for his very interesting communication, observed that the novel method of crossing wide waterways described in the Paper was possibly capable of further development. Its salient feature was the avoidance of the expensive approaches which were necessary for most bridges crossing large rivers. He would like to ask the Author and the railway-engineers who were present how far the system might be adapted for carrying railway-trains. It would of course be impracticable to carry a whole train across on such a carriage as that used at Widnes, but it was quite possible to conceive an enlargement of the design which would render possible the transport of railway-trains in running order across a wide river. The President

The AUTHOR mentioned that since the Paper was written, and since the opening of the bridge, one or two alterations had been made, a description of which might be of interest. In the first place, some difficulty had arisen in connection with the heating of the main bearings of the trolley-motors. Various kinds of anti-friction metal were tried, and some improvement was made, without, however, quite remedying the defect, and forced lubrication had to be adopted. The trolley was articulated into five sections, and each section was originally designed to be connected with the contiguous section by a single pin. It was found that, on account of the deflection of the girder from the passing load and local deflection, the section of the trolley which carried the motors was getting more than its share of the load, the joints of the trolley-sections not being sufficiently yielding vertically. The single pins were therefore dispensed with, and double pins with coupling-plates were substituted for them, with the result that the load on the bearings was now more evenly distributed, the bearings ran perfectly cool, and forced lubrication was not necessary. In designing the trolley-wheels, one point had been overlooked. It would be seen from *Figs. 31* that the centre of the tread of the wheel was not in line with the centre of the bearing; the position of the tread of the wheels was fixed by the amount of clearance when passing the The Author.

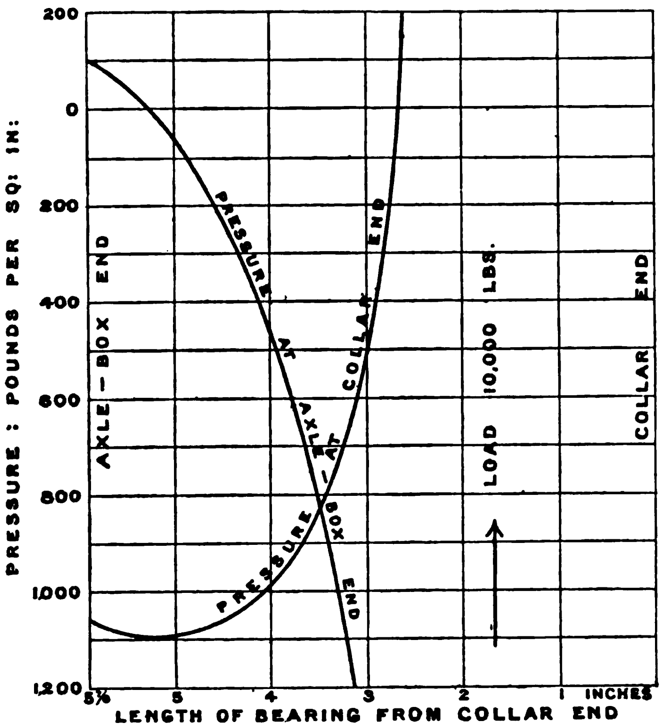
The Author. rocker, and any increased length of bearing could be obtained only on the one side. The bearing had been made as long as possible in order to keep down the unit-pressure, and thus had arisen the unequally loaded bearing. As there was more tractive resistance offered than had been anticipated, upon calculating the actual pressures on the bearing it was found that the maximum pressure was at the collar end of the bearing, and that the pressure gradually decreased—as shown by the ordinates marked A in *Figs. 31*—until it reached zero at a point about 1 inch from the axle-box end: beyond this point—as shown by the ordinates marked B, the pressure became an increasing negative pressure; that was to say, for this short length the bearing was acting as a brake. *Fig. 32* showed the pressure-curves on different lengths of bearing measured from the collar end, where a load of 10,000 lbs. was assumed to be applied in the direction of the arrow in all cases, this position being the exact relative position of the load on the tires in the actual structure. The ordinates gave the unit pressure on the bearing at the collar end and the axle-box end of the bearing; for example, if the bearing were 4 inches long, the collar-end pressure was 1,000 lbs. per square inch, and at the axle-box end it was 430 lbs. per square inch. It would thus be seen that, for the two pressures to be the same, the length of the bearing must be 3½ inches, when the pressure over the whole bearing would be uniform. By reducing the bearings to this length, although the maximum pressure was reduced and the negative pressure eliminated, the average pressure was increased. In order to overcome this difficulty, more wheels had been fitted to the trolley, and instead of having them running on a single fixed bearing, bogie-wheels with spring attachments had been adopted. The Author then exhibited a number

Figs. 31.



of lantern-slides illustrating the progress of the construction of the bridge; also a working model of the bridge, a full-sized section The Author.

Fig. 32.



of the cable, a sample of one of the strands, tested samples of wire, and the pins referred to in the Paper.

Mr. W. B. Worthington mentioned that about 20 years ago he had charge, as the Divisional Engineer on the London and North Western Railway, of the inferior mode of crossing then in use, namely, the ferry, the approach to which had to be maintained by the railway-company. It was obvious that the excellent means of crossing provided by the Author's bridge had greatly increased the traffic, which showed how the provision of facilities fostered traffic. It would be interesting if the Author could give particulars of any observations he had made on the deflections as the load passed across, as that point had some bearing on the President's suggestion that the design might develop into one for transporting railway-trains. Mr. Worthington.

Mr. Moore. Mr. R. St. George Moore was particularly interested in the work, as he had been considering for some time the question of constructing a similar bridge for crossing the estuary of the Ribble. In 1899 he visited Portugalete with Mr. J. T. Wood, of Liverpool, the Author's partner in the work under discussion, to inspect the bridge Mr. F. Arnodin had built there, and which had been in use for about 6 years, though at that time, with the exception of the small bridge across the Devil's Dyke at Brighton, the system was unknown in England. The many details given in the Paper were of the utmost value to anybody who was considering the subject. He himself had prepared designs for a conveyor-bridge of 500 feet span, in which the conditions differed somewhat from those of a bridge of 1,000 feet span. After much thought, he had come to the conclusion that an ordinary suspension-bridge was not the best form where the rolling load was out of all proportion to the permanent load; and the whole question of deflection had seemed to him to be so complicated that he had chosen a braced-chain bridge. A rigid girder-bridge in some form, he found, had been adopted by French and by American designers, and an example was afforded by the conveyor-bridge over the harbour at Marseilles. So that, while three or four designers had adopted a very rigid form of girder, the suspension-bridge had been adopted only by Mr. Arnodin and by the Author. Another reason for not adopting the suspension-bridge was that unless the stiffening-girder was strong enough to carry the load when it was suspended from the joint in the centre and from the end, the apportionment of the stresses between the stiffening-girder and the cable was absolutely indeterminate. If the girder was stiff enough to carry the load when supported at those two points the stresses could be determined as shown by the Author on p. 102, but then there was a concentration of deflection at the centre; the result being that a suspension conveyor-bridge must have either indeterminate stresses or concentrated deflection when the load was passing the centre. He gathered from what the Author had said that deflection had given rise to considerable trouble in the passage of the car; at times the load was unequally distributed on the wheels, with the result that it had been necessary to increase the flexibility of the trolley. If the flexibility was increased, the question arose—What happened to the suspension-cables of the car? Any divergence of the travelling car from the straight line must mean a difference in the tension of those cables; and he would like to know how that distortion was provided for in the car, and whether the suspending cables were of such strength that some of them could carry the load without the support of others rendered slack by distortion.

Mr. JAMES C. INGLIS questioned whether the fact that only a single line of communication had been provided at a cost of £133,000 did not suggest that the limit of development of the method had been reached. It might perhaps be possible to increase the capacity of such bridges by an adaptation of the mono-rail idea, so as to have two cars passing simultaneously in opposite directions, though of course there would be difficulties due to the swaying of the cars in a wind. He did not see why such a structure could not be made to carry a train; for there was no reason why the ingenious trolley on the bridge should not be extended and, instead of five sections, have fifteen, in order to carry a long train. But before such a sum as £133,000 was spent in bridging an estuary in this way, consideration would have to be given to the point whether a more frequent service would not be required ultimately; and if so whether a permanent structure should not be obtained, with its approaches, for even a much larger sum of money. The Great Western Railway might one day have to face the problem of crossing the Severn. Obviously, if an arrangement of the kind described in the Paper could be adopted there, it would save a great deal of capital expenditure in the first instance; but he doubted the wisdom of adopting an arrangement which would involve limitations like those of the Widnes and Runcorn bridge. A similar bridge was approaching completion at Newport, Mon. He did not know what the intentions of the Corporation were, but an advance might be made in the direction of carrying trains by taking the tram-cars across, and perhaps Sir William Thomas Lewis could say whether that was contemplated. The Paper was particularly interesting, inasmuch as the information given was very complete and the difficulties as well as the successes had been explained. He had been especially interested in the point with regard to the pin coupling, because it had occurred to him that with a single pin there would be difficulty. Mr. Inglis.

Sir WILLIAM THOMAS LEWIS, Bart., stated that he knew nothing about the Newport Bridge, or he would have been glad to give the meeting any information he could. Sir William T. Lewis.

Mr. A. T. WALMISLEY thought that if the system was to be adopted in any way to carry a railway-train, or any portion of a passenger-train, the mode of suspension would have to be altered. He desired to know what amount of deformation of the cables the Author had found with the travelling load. Certain satisfactory results had been obtained from a model, but in such experiments there was an absence of lateral pressure produced by wind. In order to adapt the method to carrying trains it would be necessary to introduce a system that would prove as effective as Ordish's Mr. Walmisley

Mr. Walmisley. system of rigid suspension, which had been adopted for the bridge over the Moldau at Prague, for a bridge at Singapore, and for the Albert bridge over the Thames at Chelsea, where the cables were prevented from deformation due to variation in the position of a passing load by a system of vertical bracing. The late Mr. W. H. Barlow had calculated that that system of rigid suspension was economically applicable to spans of 1,100 feet—100 feet longer than the Widnes bridge—so that with a braced girder no doubt greater rigidity would be obtained than appeared to be obtainable in the system adopted.

Dr. Stanton. Dr. T. E. Stanton could not agree with the Author in his description of the records of wind-pressure and velocity from Bidston Observatory as being totally unreliable because in any two of them, although the velocity recorded was practically the same, the pressure in one was 40 per cent. higher than in the other. Before accepting the Author's conclusion it should be borne in mind that the records were records of two instruments which were necessarily placed some distance apart. The velocity was measured by Robinson cups and the pressure by an Osler anemometer. He did not know what the actual distance was, but it would probably be about 5 feet, and might have been 15 feet. If it were true that in a gale the velocity of the wind at two such points was even approximately the same, the criticism would be just; but it was not certain that at any instant the velocities at two points 5 feet apart were equal. Mr. Dines had experimented 12 years ago by putting up two sensitive anemometers, 11 feet apart, and taking a series of observations, with remarkable results. It was found in many cases that one of the instruments would register 25 per cent. above, and in some cases 25 per cent. below the other. Recently Dr. Stanton had been carrying out similar experiments at the National Physical Laboratory. He used similar instruments, but put them 18 inches apart, and took simultaneous readings; and while in the majority of cases the readings agreed, still very considerable differences did occur. Therefore he thought it was practically certain that in an ordinary wind this considerable difference of 25 per cent. did frequently exist. Applying that reasoning to the Bidston records, the discrepancy was fairly easily explained. The two velocities were practically equal, but the two pressures differed by 40 per cent. Assuming that, instead of the two velocities at the pressure gauge being equal, one was 20 per cent. in excess of the other, then the pressure would be approximately 40 per cent. more on the one, which was the error indicated in the Table. He thought it likely that the rejected observations practically

represented the truth, when the circumstances under which they had been taken were considered. In his own work at the National Physical Laboratory the difficulty was accentuated, because a plate 100 square feet in area was being used, and it was impossible to measure the velocity anywhere near the plate: the instrument had to be placed 15 feet from the centre of the plate in order to estimate the velocity, and the experimenters were in the habit of getting results differing from each other by the amount he had mentioned. They felt confident, however, that by taking enough observations, say, two, three, or four hundred, an approximately true relation between the velocity and pressure would be arrived at. Dr. Stanton.

Sir Benjamin Baker.

Sir BENJAMIN BAKER, K.C.B., Past-President, had no doubt that sufficient experience had now been gained in different parts of the world to show that there was a future for transporter-bridges, where the circumstances were suitable. With regard to Dr. Stanton's remarks, naturally any engineer who took up the problem of making the lightest possible superstructure for carrying a car would consider wind-pressure, and by adopting the Board of Trade allowance of 56 lbs. per square foot and using a factor of safety of 4, he would find that the wind was of more importance than the load. But in practice it was otherwise. In the Menai suspension-bridge, which had been standing for so many years, the floor originally consisted simply of timber planks without any wind-bracing. On being called in some years ago to advise he had put in a steel floor; but he had been unable to find any evidence that the absence of wind-bracing interfered with the traffic, even in considerable gales. There was a vertical undulation —a kind of wave along the bridge—but no lateral oscillation; the result of the undulation was that a number of the suspension-rods had been broken at different times, but, as they were only 1-inch square bars, that did not imply any great force. The effect of wind was therefore not what a student would expect if he proceeded on the ordinary data given by text-books. With regard to the system generally, the most primitive mode of crossing a river was of course by a ferry-boat. Where the range of level of water was considerable, on account of either tidal influence or floods in the river, the slope of the foreshore caused great inconvenience in regard to approaches, carts having to go down the foreshore and foot-passengers having to wade through the river-mud along the causeway; to meet that difficulty ferry-boats were constructed with a deck which could be raised so as to keep it always at the level of the approach. More than 50 years ago a Past-President of the Institution, Robert Stephenson, constructed such a ferry-boat for carrying railway-carriages and other things across the Nile at Cairo, and it was in use for many years. Before

Sir Benjamin Baker. the insurrection of Arabi and the bombardment of Alexandria some of the officers of the Intelligence Department at the War Office came to him to know if the ferry-boat was still in existence. As it had been broken up about 15 years previously, their information was not quite up to date. Such a ferry was now working across the Clyde at Glasgow. Even ferry-boats with raised decks, however, were still subject to the nuisance of the current, which made it difficult to fetch the opposite landing-stage in a fog. Consequently, a transporter-bridge was an improvement, because it gave the same facility in respect of approaches, and the difficulty caused by the current—and to a considerable extent also that due to fog—was overcome. But that method would suffice only where the traffic was not excessive, because, as Mr. Inglis had pointed out, the accommodation was not nearly so good as that of an ordinary bridge. Quite recently he had had to consider a number of alternative schemes for the traffic between Barrow-in-Furness and Walney Island, and he had submitted transporter-bridges and ferry-boats with raised decks; but none of these would meet the case, for the same reason that railways in London had great difficulty in dealing with the suburban traffic—everybody wanted to travel at the same time. After going into the matter in great detail with the local authorities the conclusion had been come to that a transporter-bridge would be useless where thousands of workmen wanted to cross to their dinner at the same hour, with only limited time at their disposal for getting to and fro. The result was that a fixed bridge with opening spans had been found to be well worth its cost, and there was hardly any limit to a bridge with opening spans if the money was available. At Detroit they were once contemplating a bridge with an opening span of 600 feet, but he fancied now they would substitute a tunnel, which was another way of getting continuous communication across a river. But there were many cases in existence, and there would be more in future, where bridges of the kind described by the Author would be the proper solution of the problem, and he was glad to see that the conditions had been met in every respect by the Author, who deserved great credit for the way in which the work had been carried out, and for the manner in which the details had been placed before the Institution.

Mr. Blount. Mr. BERTRAM BLOUNT pointed out that the particular interest of the specimen illustrated by *Fig. 22a*, lay in the fact that it clearly displayed the nature of the metal. The bolt was as homogeneous as mild steel usually was. He had been taken to task at least once in the Institution for denying the homogeneity of mild steel; but he thought that any impartial person who examined

a photograph of that bolt would admit that when he stated, a year or two ago, that mild steel was a bundle of muscles and nerves, he was not far wrong. He would like very much to know what was the remarkable substance found when cutting the foundations for the piers—the curious yellow substance said to have the consistency of white lead. He congratulated the Author on his excellent plan of abolishing the absurd practice of leaving the metal-work exposed in the anchorage-pits. Metal embedded in good concrete might be dismissed from further consideration; it would remain unchanged until the whole structure perished. It was known to many engineers, but was not so generally realized as was desirable, that metal structures—at all events steel structures, which mostly concerned engineers—were permanent in concrete; the iron would not rust or decay. It delighted everybody who knew the difficulties of the matter to know that the National Physical Laboratory had not relaxed its efforts to determine so important a set of data as the facts of wind-pressure. If the Laboratory confined itself to the determination of such data it would be performing services of greater public utility than some services which it seemed to be intent on rendering in other directions. Mr. Blount.

Mr. A. S. E. Ackermann understood from hearing the Paper read that the load had been put on the anchorage before the concrete round it had been put in. He recognized the advantage it would be to do this if it were possible, but he did not see how it could be possible. He noticed that several curved members had been used in the structure, an example being the top and bottom booms of the second cross girder from the top, between the towers. That might have been done for the sake of appearance, but it seemed very faulty design, and he would like to know why it had been adopted. Mr. Ackermann.

Mr. R. J. G. Read asked the reason for coating the anchorage-bars with tar-varnish before concreting them. According to his experience, extending over several years, steel embedded in concrete, if put in without any paint or covering whatever—and even in a rusty condition—was preserved perfectly intact and bright. It seemed to him that any coating like tar-varnish or paint would only prevent the concrete from adhering to the steel and would be detrimental instead of advantageous. The same remark applied to the cables. The Author described the cables as being coated with a bituminous compound and covered with canvas, but it seemed to Mr. Read that if concrete or cement was good for embedding the anchorage it would be good also for covering the cables. When in New York he had inspected the old Brooklyn bridge, and had Mr. Read.

Mr. Read. been struck by the covering of the cables, which appeared to be a kind of cement grout or fine concrete. He did not know whether underneath that there was a bituminous compound, but certainly the outside had the appearance of a rigid concrete bar. The erection derrick seemed to have been very suitable for its purpose, the height being moderate. In the case of high towers, such as the Eiffel and Blackpool towers, the crane and erecting apparatus usually travelled up the tower itself as the construction progressed, which was about the only practicable method.

Mr. Whyatt. Mr. H. G. Whyatt had tried tar-varnish both buried and above the ground as a sort of skirting, and he had found it very unsatisfactory; it did not protect metal-work at all in that position. Had the Author used it previously, or was this his first experience of it?

Mr. Bernays. Mr. J. Bernays could not quite see how the stiffening-girder was prevented from shifting longitudinally. If he understood aright it was carried at both ends by movable struts, but he had not come across anything in the Paper to show how the girder was prevented from moving longitudinally, because the suspension-rods could offer very little resistance to horizontal movement of the girder.

The Author. The Author, in reply, remarked that there would be no difficulty whatever in giving effect to the President's suggestion to adapt the system to the carrying of railway-trains; it was simply a question of designing the car and trolley to accommodate the train, and these could be made to carry any load. The question of the amount of the actual deflection of the girder with the passing load, referred to by Mr. Worthington, was a most interesting one, and the results of the Author's observations were given in Tables I and II. With regard to Mr. Moore's remarks, before the construction of the bridge at Portugalete by Mr. Arnodin 6 years ago, the first design for a transporter-bridge, as stated in the Paper, was made 35 years ago for crossing the River Tees at Middlesbrough; the bridge, however, had not been erected. The transporter-bridges abroad referred to by Mr. Moore which had rigid girders were all of much smaller span than the Widnes bridge, and the Author himself would not recommend a suspension-bridge under such conditions. The braced-chain mentioned by Mr. Moore would not, however, as he supposed, eliminate the question of deflection; and his statement that the stresses in the cable and the stiffening-girder were absolutely indeterminate unless the girder was strong enough to be supported at the end and at the centre pier, was hardly correct; for the deflection of the girder so supported could be determined, and the correction made for the uniform pull of the suspenders, which could be ascertained

The Author.

TABLE I.

Amount and Position of Load, including Trolley, Car and Traffic.	Vertical Displacement of Girder at			Remarks.
	Centre of Widnes Girders.	Central Hinge.	Centre of Runcorn Girders.	
	Feet.	Feet.	Feet.	
75 tons at centre of Widnes girder . .	−1·17	−0·55	+0·05	Negative sign denotes downward displacement. Positive sign denotes upward ditto.
75 „ „ central hinge	−0·46	−1·73	−0·46	
75 „ „ centre of Runcorn girder .	+0·05	−0·55	−1·17	
90 „ „ centre of Widnes girder . .	−1·25	−0·57	+0·06	
90 „ „ central hinge	−0·50	−1·90	−0·50	
90 „ „ centre of Runcorn girder .	+0·06	−0·57	−1·25	
110 „ „ centre of Widnes girder . .	−1·50	−0·80	+0·04	
110 „ „ central hinge	−0·72	−2·42	−0·72	
110 „ „ centre of Runcorn girder .	+0·04	−0·80	−1·50	

TABLE II.

Amount and Position of Load, including Trolley, Car and Traffic.	Actual Deflection of Girder at its Centre.		Remarks.
	Widnes Girder.	Runcorn Girder.	
	Feet.	Feet.	
75 tons at centre of Widnes girder . .	−0·89	+0·32	Negative sign denotes downward deflection. Positive „ „ upward „
75 „ „ central hinge	+0·38	+0·38	
75 „ „ centre of Runcorn girder .	+0·32	−0·89	
90 „ „ centre of Widnes girder . .	−0·96	+0·39	
90 „ „ central hinge	+0·45	+0·45	
90 „ „ centre of Runcorn girder .	+0·39	−0·96	
110 „ „ centre of Widnes girder . .	−1·10	+0·45	
110 „ „ central hinge	+0·50	+0·50	
110 „ „ centre of Runcorn girder .	+0·45	−1·10	

The Author. from a diagram similar to one shown in Figs. 12, Plate 2. There was no difficulty in dealing with the deflection at the centre of the span, the rails being raised at this point to counteract it; and in order to allow for the unequal pull on the car-suspenders due to the curvature of the girders during the passing load, helical springs were attached to the bottom of each suspender, and the car itself was not absolutely rigid longitudinally. Judging from Mr. Walmisley's statement that it would be necessary to introduce Ordish's system of suspension for heavy loads, such as a train, he appeared to have a misconception of the action of a stiffening-girder, of which the finest and most recent example was in the new East River bridge at New York, which carried two elevated railway-tracks, four trolley-car tracks, roadways and footways. For such a bridge the Ordish system was impracticable, but the stiffened suspension-bridge gave all necessary rigidity. Notwithstanding Dr. Stanton's very interesting explanation of the possibility of wind-gauges recording different results when placed near to one another, and admitting such a possibility, it was only by assumption that the recorded differences referred to were caused by the action he described; and in the absence of any proof of the accuracy of these instruments when under very high pressures, the Author still felt justified in doubting their reliability. Apparently Mr. Ackermann had not quite understood the statement in the Paper about putting the concrete in the anchorage after the load was applied; this of course referred to the concrete put in to cover the links, not to the concrete supporting the load. As to his objection to the curved members in the bracing between the towers, and the reason why it had been adopted, although theoretically the line of stress between two points was of course straight, with an elastic material it was not necessary for this line to fall within the plane of the material; and so long as the member was properly designed to withstand a given stress, the shape of it was quite immaterial; in the present case, a curve had been adopted, principally for æsthetic effect, the towers and other portions of the structure being curved for the same reason. In reply to Mr. Read's question about tar-varnishing the links before the concrete was laid, this had been done to protect the links from the weather during many months of exposure before the concrete was laid; it is not necessary for the concrete to adhere to the links in order to protect them; but as a matter of fact, the tar-varnish did not prevent it from doing so. Mr. Read's suggestion as to covering the cables with concrete was impracticable; the working of the cables would very soon disintegrate the concrete, and it was for this reason that the more plastic bituminous

compound had been adopted. Mr. Whyatt's experience with tar-varnish appeared to have been unfortunate; and it suggested that the material was either faulty or improperly applied, for the Author has used it extensively for many years on outside work, such as iron piers, bridges, etc., with great success. Before selecting the material to be used to coat the structure, he had a dozen wrought-iron plates, 12 inches square, covered with two coats of different compounds, and suspended them about 50 feet above some chemical-works in the district for 3 months; this was a severe test, as the atmosphere was heavily laden with chemical fumes. From the results obtained, he decided upon the paint or compound to be used for the various parts of the structure. The longitudinal movement of the stiffening-girder was checked, as stated in the Paper, by the adoption of raking guys 3¼ inches in diameter, fixed between the centre pin and the cables. The Author.

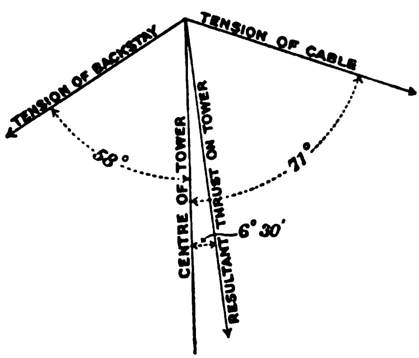
The PRESIDENT remarked that deposits corresponding in appearance with the one alluded to in the Paper were found in many places in the neighbourhood of lakes and estuaries, especially in the north of Ireland. The material had the appearance of clay, but was entirely composed of diatomaceous material; it was used largely in the preparation of explosive compounds such as nitro-glycerine, also for non-conducting covering for boilers. The President.

Correspondence.

Mr. F. ARNODIN, of Châteauneuf-sur-Loire, observed that the Widnes and Runcorn transporter-bridge was a remarkable work in more than one sense. The fact that this bridge was the longest suspended span in the United Kingdom rendered it the more regrettable that in its construction the improvements realized in transporter-bridges erected in other countries had not been introduced. In the following critical examination of it questions of details and cost of construction would be neglected, and only defects which might have an influence on the security and life of the work would be considered. Judging by Fig. 1, Plate 1, the back-stays were inclined at an angle of 58° to the vertical, while the tangent to the parabolic cable was inclined at an angle of 71°. The resultant force due to the tensions in these two directions acted along the bisector of these combined angles, that was Mr. Arnodin.

Mr. Arnodin. to say, along a line inclined at 6° 30′ to the vertical (*Fig. 33*). As this line passed outside the base of the towers, it followed that these were subjected to an overturning tendency towards the river, and that the supports on the river side were stressed much more than those on the land side. In order to remedy this the saddles which carried the cables (Fig. 2, Plate 2) had been placed on an oscillating sector, and the cables rested in a groove on the saddles, where they were clasped by jaws in order to prevent them from slipping. This was not a happy arrangement, for the single cable was composed of nineteen elementary strands, in contact at points of the circumference, which left between them very appreciable voids. The grouping of these elements could not then be considered as forming an immovable and incompressible whole. By the tension as well as by the dilatation, the points of contact were indented under the pressure, and the jaws lost little by little the energetic grip which in principle should hinder the slipping. It followed that the slipping was always to be feared, and if it occurred it might bring about the ruin of the whole structure. Besides, the back-stays being more inclined than the cables suffered an extra tension of about 12 per cent. As the cables were of uniform section throughout their entire length, the back-stays were subjected to a stress 12 per cent. in excess of that in the parabolic portion. From this it appeared that there was either insufficient section in the back-stays or excess of section in the parabola. It was to obviate these two inconveniences that in all the other suspended transporters in the world a junction had been made over a gudgeon on the top of the pillars, in such a way as to prevent absolutely all possible slipping, and to enable different sections to be adopted for the back-stays and cable, so that they might sustain the same stress. Fig. 7, Plate 2, showed that the elementary strands, to the number of nineteen, were grouped in hexagonal form and surrounded with a common envelope serving for protection against oxidation. Experience had proved that in this kind of cable, whatever steps might be taken, there always existed interior voids

Fig. 33.

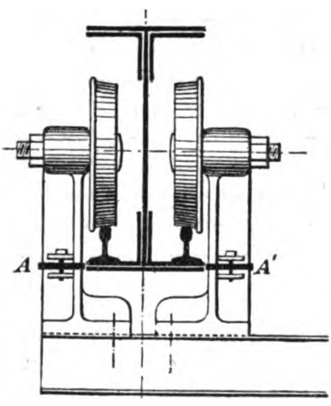


Mr. Arnodin.

wherein rain-water accumulated by percolating through imperceptible fissures in the envelope. The water flowed in these voids as in so many tiny canals, carrying deterioration into the centre of the bundle. This deterioration might not begin for several years if the preservative coverings had been applied with the greatest care; but when it began it made rapid progress, for as it was hidden from inspection it could not be counteracted, and a cable of this kind was thus doomed to comparatively rapid destruction. It was for the purpose of avoiding this defect that in France it was now the practice to keep the cables separate from one another, in order to be able to inspect and paint them when necessary. In connection with the preservation of large suspended works French engineers had gone farther, and the French Minister of Public Works prescribed in the regulations for new works the principle of "interchange" which Mr. Arnodin had introduced into all his suspension-bridges. This principle consisted in arranging the members in such a way that it was possible to displace any piece whatever, repair it or change it for a new one, and replace it in its original position, without being obliged during the operation to displace the neighbouring pieces, or even to interrupt the normal service of the work, the strength of which was quite sufficient when depending on the fellow-members of that which had been displaced. For this purpose members of the same kind were built in multiple, there being at least three side by side. According to this principle of interchange, when a cable deteriorated in the course of a number of years, it would be replaced by a new one without having to demolish anything; and by repeated replacement it would be possible to make the suspension-bridge last indefinitely, just as a railway was maintained. The Newport transporter-bridge, which was now being constructed by Mr. Arnodin in collaboration with the borough engineer, Mr. R. H. Haynes, was the first work in the United Kingdom constructed on this principle, although it had been applied in France for the last 10 years. In Figs. 24 and 26, Plate 3, the trolley from which the transporter-car was suspended was shown running on rails placed on the side of the lower flange of the main girders. This was a bad arrangement, for it developed in the girders an unsymmetrical stress which tended to tilt the girder, and when the girder became inclined the wheels tried to leave the rail. This might cause a grave accident, for the flanges of the wheels were insufficient to prevent derailment. In the transporter-bridges which had been recently constructed, *e.g.*, those at Rouen, Martrou, Nantes, Marseilles, care had been taken to put symmetrically on each side of the web of the girder a double line of rails, as in *Fig. 34*, in such

Mr. Arnodin. a way as to exert only symmetrical stresses, and not to produce any tendency to distortion, whatever might be the load carried on the transporter-car. There was a gain in having the wheels in duplicate for the suspender from the car, for it permitted the removal of either of the wheels even when working, without any fear of the corresponding cable being slackened. By placing small horizontal wheels at A and A′ an arrangement was obtained which absolutely resisted any tendency to derailment. The conditions were then superior to those of ordinary wagons on a railway, which were liable to be derailed, and whose wheels it was impossible to replace during their normal work. There was an advantage in putting a number of wheels to bear the transporter-car in order to obtain a relatively light load on each; and in order not to fatigue the angles of the main girders which carried the rails, the length of the car and the obliquity of the suspenders which supported it necessitated the extension of the framework of the trolley to a considerable length. It followed that if axle-motors had to be employed it was necessary to use many, otherwise the motor-wheels would slip on the rails without causing movement. Under normal conditions the four motor-axles provided at the Widnes and Runcorn bridge were probably sufficient for the traction, but when a gale was blowing, or when the car was heavily loaded, the traction would be difficult; for, as the weight in the car increased, the flexibility of the suspension-bridge increased, and the resistance to traction became very appreciable. Now, as the motor-axles were placed at one-quarter and three-quarters of the length of the trolley, it resulted from the deflection of the bridge under the load that the extreme wheels carried proportionally more than those of the motor-axles, and the latter were consequently more liable to slip. Lastly, the slipping of the motor-wheels wore them more than the free wheels, and this wear still further diminished their power of adhesion. These different phenomena had been well understood by the constructors of all the other transporter-bridges in

Fig. 34.



the world, including that at Duluth, Minn., for all of them had preferred to haul the trolley by means of a cable, with wheels turning freely on their axles. All the wheels were then in the same condition of rolling as those of a wagon hauled by a locomotive. Besides affording security of travelling during stormy weather, this arrangement had the advantage of requiring less motive-power. Mr. Arnodin.

Mr. C. H. Colson observed that it would be interesting if the Author could describe more in detail the methods adopted to form the cable from the separate strands, and to ensure that each strand took its proper share of work and was in exactly its right position. The reasons which had led to the steelwork in the anchorage being coated with "several coats" of tar-varnish were not clear. The numerous experiments made during the last few years on the corrosion of steel when embedded in cement concrete had proved fairly conclusively that clean steel embedded in non-porous concrete was quite unaffected by corrosion, and that even if the steel were slightly rusty before being placed in position, no ill-effect was to be feared. The protective action was said to be due to the formation of a silicate of iron; but if the steel was painted or covered with tar-varnish before being placed in the concrete, this silicate could not be formed; hence if the tar or paint coating was at any time destroyed, a void would be left into which water might penetrate and set up the corrosive action that would have been prevented had the concrete been in actual contact with the iron. For this reason it appeared to be unfortunate that the anchorage-links had been coated with any paint or tar composition. In Mr. Colson's practice the steel bars when brought on to the works were reasonably cleaned of all rust and coated with thick cement grout applied as paint; this appeared to stop further corrosion and to keep the bars in good condition. It was perhaps doubtful whether the concrete in the cylinders was sufficiently rich in cement to ensure its being water-tight. Some time ago he had the opportunity of inspecting the remains of ferro-concrete piles which had been cut off during the construction of a jetty and left for 5 or 6 years on the foreshore, covered by the sea at every tide. On breaking open the pile-ends it was found that, where covered by the concrete, the bars were in quite good condition, whereas the ends projecting outside the concrete were almost eaten away by rusting. In view of the small size of the tower-cylinders it would be useful to know what considerations had governed the selection of the pneumatic process for sinking them instead of the method of excavating under water inside the cylinders by grabs. The latter method would have been cheaper and would have allowed of continuous working. Mr. Colson.

Mr. Duryea. Mr. EDWIN DURYEA, jun., of San Francisco, whose experience with suspension-bridges consisted of about 6 years' work in various capacities in connection with investigations for, or the design and construction of, several large suspension-bridges, including the Williamsburg bridge, New York City, of 1,600 feet span, the old Brooklyn bridge, of 1,596 feet span, and the proposed Hudson River bridge, at 59th Street, New York, of 2,850 feet span, remarked that the discussion of the Paper might be divided into two parts; first, the uses for which the bridge should have been or had been built, and the completeness with which the special structure under discussion provided for them; and secondly, the structural features of the bridge. In respect of its object, which was to provide for pedestrian and vehicular traffic only, the bridge was similar to one recently completed in the United States at Duluth, at the head of Lake Superior.[1] In that case the only object secured had been pedestrian and vehicular traffic between the city of Duluth and Minnesota Point, a narrow spit of land more than 6 miles long and only a few hundred feet wide, separating Lake Superior from the Bay of Superior, which included Duluth Harbour. A much more important object, that of giving direct railway connection between the mainland and Minnesota Point, thus adding 6 miles of much needed dock-room to the harbour, had been entirely neglected; though the building of a tunnel about 2 miles down the Point, the subsequent dredging of a canal over this tunnel from the lake to the harbour, and, finally, the filling-in of the present canal (all of which had been suggested some years ago) would at the same time have given this additional dock-room and greatly improved the harbour in other ways. It was not apparent, however, that any such additional benefit could have been secured in the case of the Widnes and Runcorn bridge by a more positive connection of the two sides of the river; and if no such larger need really existed, the present structure should serve all purposes. At the same time, as a permanent carrier of general traffic a transporter-bridge was at best only a makeshift, in comparison with an unbroken roadway; the cost of the structure was £133,000, or about $650,000, which was a large sum; and fuller explanation seemed necessary to show why the additional cost of a bridge or tunnel with an unbroken roadway would not have been well expended, or why the unbroken roadway might not even yet be needed in the near future to provide for increase in traffic, or for classes of traffic which could not be adequately served

[1] C. A. P. Turner, "The Ferry Bridge across the Ship-Canal at Duluth, Minnesota." Trans. American Society of Civil Engineers, vol. lv, p. 322.

by a transporter-bridge. With regard to structural design, the choice of a suspension-bridge seemed a wise one in view of the comparatively long span (1,000 feet), the rock anchorages which were economically available, and the very light moving loads. Although the Author did not state it clearly, the maximum total moving load on the span—the transporter-car with its trolley, hangers and traffic load—might be inferred to be only 120 tons, or only 269 lbs. per foot of span. The Author was to be commended for his choice of a suspension-bridge, but not for his treatment of the subject in his Paper. The most severe criticism on the Paper, from the standpoint of an engineer who had had to do with suspension-bridge design, was that it dealt principally with details, and almost entirely neglected to treat the methods used in the design. These were of special interest in connection with suspension-bridges, as no standard methods of design and computation for such structures seemed yet to have been agreed on by the few engineers who had had to deal with this form of construction. The value of the Paper would be greatly increased if much fuller data were given as to loads, methods, assumptions, formulas, and results. At present it contained very little which dealt with the actual design of suspension-bridges. The characteristic elements of a suspension-bridge were the cables, the anchorages and the stiffening-trusses. The towers were of equal importance, but had no special relation to the suspension principle. The platform and its suspenders were subordinate features. The ideal suspension-bridge, it was believed, should have as few cables as practicable—preferably only two, unless they would then become too large—and should have only two stiffening-trusses. The tower at each end of the span should have not more than four posts, or preferably only two, and these arranged to rock at the base. Cables should preferably be "straight" in the shore spans, but if it was desirable or necessary that the shore spans should carry loads, their cables should then be anchored near midspan by anchor-bents to limit the deformations of the cables. In general, it was unnecessary and unadvisable to cradle the cables. The bridge under discussion conformed to some of the above requirements, but to others it did not. The sag of the main cable was about 84 feet, or about one-twelfth of its span. This ratio was too small for bridges of longer span and carrying heavier loads, the economical proportion for these being about one-eighth, equivalent to 125 feet for the Widnes bridge. The small sag here used seemed advisable, however, because of the (for a suspension-bridge) short span and the very light loads. The effect of the smallness of the sag was to increase the changes in sag arising from variations in temperature and stretch, and to decrease

Mr. Duryea.

Mr. Duryea. the deformations of the cable from moving loads and thus lessen the duty which must be performed by a stiffening-truss. It would be of interest to learn how fully the changes in the cables from variations in temperature and from deformations and stretch caused by moving loads had been considered in the design of the structure. In the cables a detail which differed from what was considered good practice in America was the fastening of the ends of each strand to the anchor-chain by means of sockets. In America, good practice had always been thought to require that the wire in each strand should be continuous, except for unavoidable splicing, and that the strand should be directly connected to the anchor-chain by a loop. Numerous tests should of course show whether socketing was safe and could develop the full strength of the strands, but the feeling among American engineers had always been against the use of sockets. No large or important American bridges had them in cables, and the best practice did not sanction their use even in suspenders. The use of a centre hinge in the stiffening-truss was very commendable. Its safe use, however, entailed some provision to limit the bending in the cables above the hinge, from changes in sag. This could easily have been accomplished by supporting the suspenders at the centre of the span on a special casting of somewhat large radius, resting on the cable. The cradling of the cables seemed advisable in this case to resist the wind-pressure, because the light weight of the structure and its small width (35 feet) compared with its span, gave it but little power to resist the wind. It did not, however, seem advisable generally to cradle cables, as the complications in the structure which resulted were not usually justified by the comparatively small good effects thus secured. The saddles at the top of the towers were unusual in only two features, neither of which was entirely new, but both of which were good. One was the use of segmental rollers, and the other was the adoption for the back cable of a different angle with the horizontal from the main cable. The last feature had been proposed by Mr. Joseph Mayer in connection with his designs for a bridge of 2,850 feet span over the Hudson River; as had been also the positive connection of the cable to the saddle, and of some of the suspender-clamps to the saddle by an auxiliary cable. The wisdom of the Author's adoption of a steeper slope for the back cable seemed doubtful, however, as greater stress in the back than in the main cables was thus caused, and hence the size of the cables was increased throughout their whole length. It would be interesting to learn whether the saving in length of the back cables and the possibly better conditions at the anchorage-pits fully compensated for the increase in cost of cables due to the steeper

back angles. The anchorages were of a usual design and simple and effective. The filling of the pits with concrete around the lower ends of the anchor-chains at once, followed later by the complete filling of the anchor-chain tunnels, was a very advisable feature and one which had been adopted in the best American practice. The eight posts at each end of the span in the Author's design had been used in America, but these were unnecessarily complicated and were adverse both to a clear determination of stresses and to economy of construction. The use of a two-post tower rocking on its base, at each end of the span, would have served all purposes, greatly simplified the structure, and saved a very appreciable part of the cost of the towers and foundations. If the rocking tower was not considered desirable, much simplification and saving in cost could still have been effected by using a four-post instead of an eight-post tower. One of the most characteristic elements of a suspension-bridge was the stiffening-truss. The design of stiffening-truss used by the Author was believed to be wise in including a central hinge, but no advantage had been taken of that hinge to adopt rational and economical proportions for the stiffening-truss. With a central hinge it was practicable to adopt the economical proportions, which were approximately those of other trusses, say a height equal to one-ninth of the span with a curved top boom, or perhaps one-twelfth with a horizontal top boom. The span here meant was one-half the cable-span, or the distance from the central hinge to either tower, namely, 500 feet. The economical height of truss would then be perhaps 56 feet with a curved top boom, or 42 feet with a horizontal top boom. The uniform height of the truss as built was 19 feet, only one-twenty-sixth of the half cable-span. If a definite degree of stiffness was to be provided by the stiffening-truss, this could have been secured much more economically by a higher truss; or with the same expenditure as for the lower truss, much greater stiffness would have been provided by a higher one. This question had been treated thoroughly and mathematically by Mr. Mayer,[1] and also, without mathematics, by Mr. Duryea.[2] It was questionable, however, whether the degree of stiffness already provided in this bridge was not unnecessarily high. In view of the unprecedentedly light moving load and the small sag of the cables, it would seem that local stiffening only might be sufficient, and the larger function of the stiffening-truss (the dis- Mr. Duryea.

[1] "The Stiffening System of Long-Span Suspension Bridges for Railway Trains." Trans. American Society of Civil Engineers, vol. xlviii, p. 371.

[2] *Ibid.*, p. 442.

Mr. Duryea. tribution of local loads over the whole cable-span) unnecessary. However, the light fixed loads, due in part to the absence of any platform or floor, tended to make the necessity for the true stiffening-truss distribution more probable. The mathematics of suspension-bridge design dealt principally with the relations between cables and stiffening-trusses, and if the Author would include a full treatment of this in his reply great value would be added to his Paper. The single, definite, short, moving load made the problem unusually simple and definite, and permitted of accurate treatment to an unusual degree. With regard to the specification, the requirements for Portland cement were lower as to both strength and fineness than those usual in ordinary American practice, and Portland cement of the qualities specified would not usually be acceptable in first-class American engineering work at the present time. The proportions specified for concrete showed the use of rather richer mixtures than were usual in American works, but the specifying of cement and aggregates only, without giving the proportions of sand and of stone or gravel in the latter, made the two practices rather hard to compare. In general, the most common American practice on first-class concrete work was 1 part cement, 3 parts sand and 5 parts broken stone or 6 parts gravel (packed volumes)—the richest perhaps 1 : 2 : 4, and the poorest perhaps 1 : 3 : 7. The specifications, as given, did not permit of machine mixing of concrete. Machine mixing was now almost universally used in America, and was believed to be in England, not only because of its cheapness, but also because better concrete was believed to result from it. The minimum elongation specified for the steel was somewhat low according to American ideas. Examination of the abstract of contract prices indicated that in general the prices for the Widnes bridge were higher than the prices would be for corresponding work in America. This might be partially due to the small quantity of each kind of work in this bridge. Mr. Duryea's general impression was that the total cost of the Widnes bridge was unduly high in comparison with the somewhat limited service, which was all that could be given by a transporter-bridge. His ignorance of local conditions, however, might render this view erroneous.

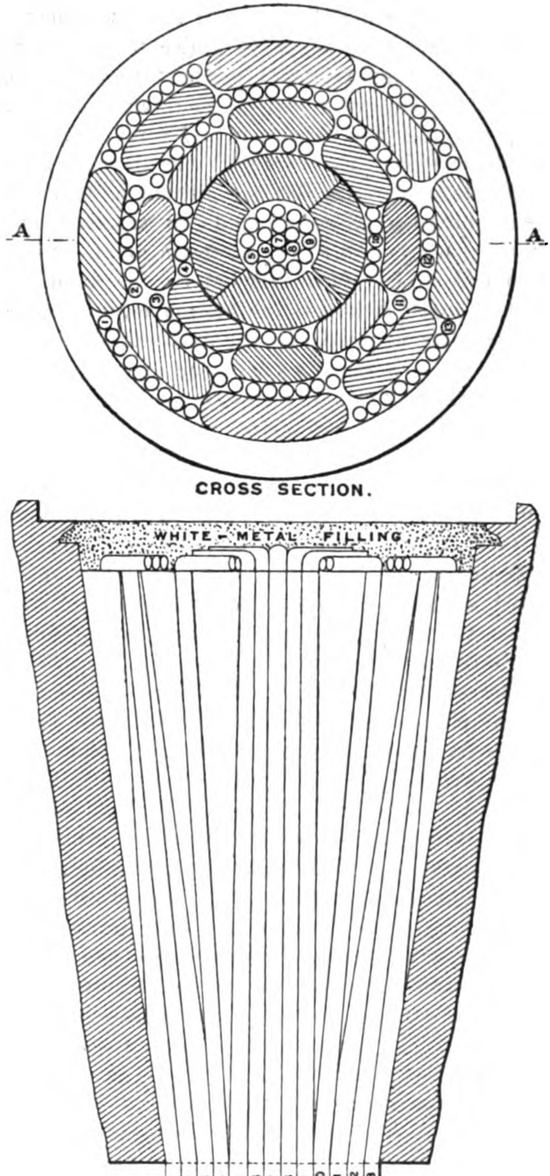
Mr. Glover. Mr. W. J. Glover, who had been responsible for the manufacture and erection of the steel cables, observed that in his opinion the design adopted for the cables presented many advantages over the selvagee form of cable, constructed by laying parallel wires together. In that form the uncertainty of obtaining uniform tension on all the wires would be increased by the action of the wind during adjustment, and any slackness appearing on compressing the separate

Mr. Glover.

bunches of wires could not then be eliminated before lowering them into their final position. The aggregate loss in strength in the selvagee form of cable when tested under the most favourable conditions in a carefully prepared straight length, amounted to 3 per cent. of that of the individual wires. This loss was inevitably increased when dealing with bunches of 127 wires, as movement was unavoidable in bringing the separate wires into position after adjusting their deflection. A distinct advantage was manifest in erecting the machine-laid section of cable, by avoiding the necessity of sending men across the span, however great. The 127-wire section, when laid in the form adopted by the Author, lost only 6 per cent. in strength, although the margin allowed in the Paper was set at the safer figure of 10 per cent. The total number of wires appearing on the surface of the cable-sections was 28 per cent., and when grouped in the saddles and clips, 18 per cent. only were visible before covering with the waterproof sailcloth double serving. Experiments upon the limit of elasticity of cable-wire, recorded by Mr. G. C. Henning, showed the remarkable figure of 95 per cent. The original specification for the first East River bridge wire called for a limit of elasticity of 47 per cent., although actual deliveries showed fairly uniform results of about 85 per cent., which was the limit accepted as obtainable on steel cable-wire of 200,000 lbs. breaking-stress per square inch of section. The difference was doubtless accounted for by results obtained with the special recording-apparatus referred to by Mr. W. Hildenbrand, and not by any increase in the elastic limit of wire produced since the first East River cables were made. It would be interesting to hear the outcome of the Author's further correspondence on this subject with Mr. Hildenbrand. The erection of the main cable-sections had been successfully accomplished by the arrangement described by the Author. The carriers were mounted on the Blondin rope and fixed to the hauling-rope, but ran free on the section of cable to be passed across the span. Upon the leading end being pinned down to the opposite anchorage, the endless hauling-rope was reversed, thus withdrawing the running clips and leaving the erected cables free. The method of fitting the sockets to the ends of the strands and the arrangement of the wedges was shown in *Figs. 35.* It consisted of a symmetrical arrangement of the smallest number of wedges to ensure an unyielding grip upon the whole of the 127 wires. By this method the wires were disposed in regular form between mild steel wedges of sufficient proportions to permit the wedges to be driven home simultaneously, and finally by the application of a drift and a heavy hammer. The ends of the wires

Mr. Glover.

Figs. 35.



LONGITUDINAL SECTION ON AA.
Scale: Half Size.

were lastly turned over the back ends of the wedges, and the whole socket was run with white metal. It should be noted that the white metal was not depended upon to take any of the strain, but merely to retain the wires and wedges in their positions, and to prevent the ravages of the atmosphere. Mr. Glover.

Mr. Robert H. Haynes, of Newport, Mon., remarked that the system of propulsion described in the Paper was entirely novel so far as its application to the working of transporter-bridges was concerned, and only experience could demonstrate whether it was likely to be as reliable under service conditions as the older and theoretically less perfect methods previously adopted in continental structures, which, except in one instance, had been uniformly satisfactory. These might be described as main- and tail-rope haulage systems, which in the three earlier examples were worked by steam-power supplied by a power-house erected at one end of the bridge. In the later structures electricity had been adopted as the motive power, the control being effected from a pilot-house erected on the transporter-car. In one instance the ends of the rope were fixed in the towers, the rope being passed thence along the underside of the bridge-platform to pulleys in the centre of the truck or trolley, the bight being in turn carried down to and wound around a drum erected on a lattice structure placed over the car-roof. The drum was driven by two 25-HP. motors with series-parallel control, the latter involving the use of only two electrical leads overhead. Electrically this method had worked well, but mechanically it had not been an unqualified success, as in the event of the rope breaking the car must be hauled to one side or other by hand. The arrangement, although inexpensive, was not likely to be repeated. In the Nantes and Marseilles bridges the winding-apparatus, which consisted of a frictionally-driven drum worked by two motors, each of 30 B.HP., was fixed at one end of the bridge, the necessary electrical leads being carried along the underside of the platform. The current was taken by under-running trolley collectors and insulated conductors down to duplicate controllers placed in the operator's cabin. Series control was adopted in these cases. Six feeder- and control-wires were used to permit of the reversal of the motors and to control the braking. There were two hauling-ropes passing over and under the bottom bracing of the girder structure, both ropes being attached to a swan-neck in the centre of the trolley and again secured to the drum in such fashion that when one rope was wound on, the other was unwound. At Nantes the original hauling-ropes were still in use after about 18 months' service, and during this period no trouble had been experienced. A maximum speed of Mr. Haynes.

Mr. Haynes. 10 feet per second was obtainable. A system which obviated the use of rope-haulage appeared preferable at first sight, but whether sufficient adhesion would be obtained between the driving-wheels and the rails during frosty and snowy weather, particularly during high winds, had yet to be proved. It might be found necessary to introduce sanding-apparatus to prevent slipping. The position of the motors high up over the trolley was not an ideal one, as renewals of working parts and the replacement of armatures, which must sooner or later be done, would probably be found a tedious task involving the use of special tackle. The track-rails would doubtless have a shorter life than where rope-haulage obtained. The braking-arrangements were more simple and adequate than with rope-haulage, and in this respect the system was advantageous. Rope-haulage was being adopted for the Newport bridge. A comparison of the results of the two methods after an extended trial under ordinary conditions would enable a more definite opinion to be expressed as to which was the better. The plan of the Widnes car differed greatly from that adopted on the continental bridges, which had cars of greater width in relation to their length, it being considered that loading and discharging were facilitated by the wider car. Where vehicles drawn by tandem teams were chiefly in use in any particular locality, the longer car was more convenient. No general rule could be laid down, as each district needed special treatment. Continental designs provided for great flexibility in the stiffening-girders and a considerable increase in the number of parts, particularly in the cables and suspension attachments, the chief objects sought being facility of erection, great range of adjustment, and comparative ease in renewal, should such become necessary. The reasons given by the Author for the adoption of single cables with a minimum number of attachments were unanswerable. The extremely moderate cost of the Widnes bridge was a matter for comment; the satisfactory nature of the foundations had doubtless in part brought this about. Speaking generally, transporter-bridges had been erected in situations involving foundation-works of a troublesome and expensive nature, the expenditure under this head representing a considerable portion of the total cost.

Mr. Henning. Mr. Gus C. Henning, of New York, considered that the structure designed and erected by the Author was very interesting and practical for its purposes, in its place; but that it would have been designed with parallel-wire cables instead of cabled ropes, had the proper information been available. The tests of cables accepted by the Author as correct and satisfactory were very misleading, because those who had made them had not been sufficiently accurate and

Mr. Henning.

particular about such matters. It had been repeatedly demonstrated in America that cables of parallel wire were much more reliable and stronger than those built up of spiral wire ropes. At p. 97 the Author stated that "experiments were made at the United States Arsenal at Watertown, Mass., in 1895, with four ropes made of parallel wires and four laid with twisted wires, each rope being constructed of thirty-seven wires of No. 8 gauge." In the reports nothing was stated about the construction of the parallel-wire cables. Careful inquiry had brought out the fact that these parallel-wire cables were held in the ordinary sockets, jammed and held in place by steel wedges driven between the wires into the lead poured in between them. As the construction of the cables was all-important, and should be precisely similar to that adopted in bridges, these wires should have been laid, under absolutely uniform stress, around two shoes, like those which connected the wires to the anchor-bars, and the two ends of the continuous wire then fastened. Pins passed through the shoes, and by these pins they should be held in the testing-machine. As this had not been done, the results of the tests could not be considered to represent correctly the properties of parallel-wire cable. The report stated, in fact, that some of the wires pulled out of the sockets, an accident which could never happen when a test was properly prepared for and carried out. It was stated in the Paper that "the results showed that the wires were much more uniformly strained in the twisted ropes than in those with the straight wires." This was a misleading statement, which would not have been made had the cables been properly constructed. Again, the statement that the modulus of twisted-wire ropes was 25,000,000 lbs. per square inch against 27,000,000 lbs. for the straight-wire rope was incorrect and contrary to experience. The stretch of twisted-wire ropes was very great, showing an apparent modulus of elasticity of not more than 20,000,000 lbs. per square inch; while that of parallel wires, when all were acting together, was between 29,000,000 lbs. and 30,000,000 lbs. per square inch, according to repeated tests. The proper sectional area to be used for calculation was that based on the total weight of wire from anchor to anchor, compared with the number of wires contained in any cable. Careful examination of twisted-wire cables and also of those built up of parallel wires, had always shown that the latter were much more uniform and reliable. This was proved by the fact that in tests of twisted-wire ropes, single wires broke first, long before the entire rope failed. Mr. Henning had never seen a twisted-wire rope fail otherwise than piecemeal, as was clearly indicated at p. 98 by the words: "In making the tests of the

Mr. Henning. rope, it was found that the outside wires always broke first, which is no doubt due to their firmer grip in the sockets, those in the centre drawing slightly." This very action made it impossible to determine the modulus of elasticity of twisted-wire rope, which was possible in the case of parallel-wire cables. The stretch observed and measured when testing the former was very considerable, and was mainly due to the rearrangement of the wires in the several strands, the outside wires being forced into the inner ones : it was not the determination of purely elastic changes of length from which alone the modulus could be determined, as was possible in properly made test samples of parallel-wire ropes. Tests made of No. 7 or No. 8 wire suitable for bridge-cables showed, as stated at p. 98, a modulus of elasticity of over 29,000,000 lbs. per square inch, with a tenacity of 90 tons to 110 tons. The yield-point, as determined by a recording instrument, was found to be at least 85 per cent. of the tenacity, and had, in hard wire, been found to be as high as 95 per cent., without showing brittleness, but with an elongation of 5 to 6 per cent. in a 12-inch length.

Mr. Hildenbrand. Mr. WILHELM HILDENBRAND remarked that the whole construction of this bridge coincided so much with his own views on suspension-bridges, that he could find nothing to criticize, but everything to praise. In fact, the plan as a whole and in detail appeared to him as if it had been designed by himself, which in the eyes of the profession might be no compliment to the engineer of the bridge, but which was certainly the best compliment that Mr. Hildenbrand could express. The proportions of the bridge were harmonious and pleasing, and the towers, while giving an impression of stability and strength, were graceful and of simple beauty, without showing any pretence or effort at producing an architectural effect. The designer of the towers and approaches had abstained from encumbering them with useless ornaments, and he had succeeded in making the structure an ornamental landmark merely by aesthetic outlines, correct distribution of masses, and good proportions. The principal features of the structural details agreed, as a whole, with Mr. Hildenbrand's practice in the construction of suspension-bridges. For instance, in embedding the anchor-chains in cement, the Author had departed from the usual European custom and had followed American examples, which was much to the advantage of the structure. To make anchor-chains accessible for inspection was an alluring illusion of safeguard, but experience had shown that the easier the access was, the less were anchor-chains inspected, the more they were exposed to corroding influences, and the quicker they became decayed. He had had occasion to open anchorages which had been standing

over 50 years, and he had found all anchor-bars embedded in cement, though only imperfectly covered with paint, in a perfect state of preservation. When building the Cincinnati suspension-bridge over the Ohio river in 1895-9, he covered all anchor-irons with a layer of paraffin in addition to two coats of red lead, and he believed that the former obviated any anxiety about oxidation going on unobserved. In order to test the protective qualities of paraffin, he had placed two pieces of steel—one painted, the other covered with a coat of paraffin —in a pot of concentrated hydrochloric acid. The first piece was destroyed and vanished in a few hours, while the second piece, after having been immersed for 6 weeks, did not show any effect of the acid on the paraffin or the steel. Of course, where exposed to the sun, or where appreciable motion took place, paraffin was not lasting; but underground, or embedded in masonry, paraffin was, in his opinion, the best protection that could be put on metal subjected to oxidation. In the cradling of the cables he recognized another familiar feature of American suspension-bridges. It was first introduced by Mr. John A. Roebling, the pioneer of cable suspension-bridges in America, in a cable aqueduct over the Allegheny River, built in 1844. Different engineers had tried to prove theoretically that cradling the cables was of no advantage to a bridge, but whoever had had opportunity to make practical observations would be convinced that the lateral stiffness of the floor was decidedly increased by drawing the cables into inclined planes. He noticed that the back cables of the Widnes and Runcorn bridge formed a larger angle with the horizontal than the main cables, which caused the stress in the former to be about 14 per cent. higher than in the latter. It was therefore a question whether it might not have been more economical to socket some of the wire ropes at the towers and to continue them from there to the anchorages with heavier ropes for supplying the back cables with the needed additional strength, without wasting material in the long span. Probably the Author had considered this question, and if he had found that the extra trouble and expense of socketing was not considerably less than the cost of giving the whole cable the required strength of the back cables, he had done well to make all ropes continuous from anchorage to anchorage. Any kind of splice was always an element of weakness which should be avoided, and one of the advantages of wire cables consisted in their continuity without intermediate splices, which advantage should not be abandoned without the best of reasons. For a structure of the magnitude of the Widnes and Runcorn bridge, Mr. Hildenbrand would probably have adopted cables of parallel Mr. Hildenbrand.

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Mr. Hildenbrand. straight wires in place of twisted ropes. While there was no objection to wire ropes, it was known that parallel lines would require 10 per cent. less material, and they certainly made a more compact and better-looking cable. The end connection of straight-wire strands with the anchor-chains was perfect, while that of wire ropes was at best clumsy, and introduced an element of weakness. The tests made at Watertown in 1895 were, to his knowledge, made with socketed ropes and strands, which were very different from a large wire cable, made in place and carefully adjusted. There was an unjustifiable prejudice among many engineers against wire cables, on account of the supposed difficulty and "impossibility" of adjusting thousands of wires to the same tension. Such a supposition was erroneous. He had had experience in the construction of large wire cables after the spin-in-the-air process, and did not hesitate to say that the wires in a cable could be more accurately adjusted than any other aggregate of bridge-members in ordinary structures; for instance, the number of eye-bars forming the bottom chord of a truss, or the diagonals of the web system. The accuracy of such bridge-members depended on the workmanship and on measurement, which were never perfect. No matter how rigid the inspection might be, there were always discrepancies, and the engineer could only trust that the stress in several tension-members pulling in the same direction was equal; he did not know it, and had no means of proving it. It was different with a wire cable. There the engineer could see the tension in each wire, and by only ordinary diligence and attention the stress in the different wires would not vary by $\frac{1}{10}$ lb., or less than 0·05 per cent. For instance, in a cable of 1,600 feet span and 150 feet deflection, a No. 6 wire, when first suspended, would have a tension of about 210 lbs. If it were given only 209 lbs. tension, it would sag 10 inches below the other wires, which was a discrepancy so great that the regulator could not help seeing and correcting it. As a matter of fact, the wires were regulated within $\frac{1}{4}$ to $\frac{1}{2}$ inch, which was between 0·0125 and 0·025 per cent.; but to allow for some want of care, and for the difficulty of regulation in windy weather, he had placed the maximum variation of tension in the single wires at 0·05 per cent. In no other kind of bridge-work was the same accuracy even supposed to exist; and further, the accuracy of the tension in the wires of a cable was not imaginary, but was just as real as the weight of any piece of steel read on a pair of scales. In conclusion, he wished to congratulate the Author and the other engineers on the successful completion of this structure which would be a lasting monument of their engineering skill and good taste.

Mr. R. S. SCHOLEFIELD inquired whether there had been any tendency for the rockers under the saddles to jam, as the distributing-girder was being built gradually in its position. Had not the increasing weight pulled the saddle over towards the centre of the span, or had the clamps that were attached to the back-stays and the saddles been successful in preventing all movements of this nature? Had any special carriage or other provision been made for the painting and maintenance of the cables and other steelwork? Were the strands of the cables tied together at shorter distances than the 6-foot spacing of the soft-steel wire bindings, and had the weather found its way within the strands of the cable between the places where it was bound? Was it not difficult to make certain that all the suspending rods of the car carried their proper proportion of the load, and would not the over-screwing of some of the nuts, or the under-screwing of others, by careless workmen, have thrown an over-strain on some of the suspenders? Mr. Scholefield.

Mr. C. A. P. TURNER, of Minneapolis, considered that the Paper could be best appreciated by those who had worked out the details of a structure of this general character. The artistic treatment of the towers and approach-spans was very pleasing indeed, and many of the features of the design for the type selected seemed to be worked out in the most creditable manner possible. The arrangement of the track on the outside of the chord of the stiffening-truss appeared to him to be somewhat open to criticism from the structural standpoint, in that the load was applied eccentrically, which could, he thought, have been readily avoided by the adoption of the expedient he had used for the transporter-bridge at Duluth, Minneapolis; and it seemed to him that the usual English practice of having riveted connections for the stiffening-truss would have resulted in greater rigidity. On the question of wind-pressure, his view was that if a structure possessed a fair degree of stiffness, the cumulative effect of gusts of wind need not be considered. In connection with the car suspended by ropes with sway-cables, the Author did not state the amount of the weight transferred to the windward side by these cables in a gale, nor did he state his experience in working the car under severe wind conditions. In designing the transporter-bridge at Duluth[1], Mr. Turner had selected the double-system, Warren type of stiff, riveted construction throughout, because the site was exposed to severe gales, and it was necessary to have continuous service. The Mr. Turner.

[1] C. A. P. Turner, "The Ferry Bridge across the Ship-Canal at Duluth, Minnesota." Trans. American Society of Civil Engineers, vol. lv, p. 322.

Mr. Turner. traveller was also of stiff riveted construction, so that the total lateral sway amounted to barely ½ inch in a 60-mile gale, and was practically imperceptible. In executing the work, the contractors had not seen fit to follow his details of truck-bearings, and the cheap substitutes furnished did not stand the wear more than a little over 3 months, when they were replaced by others. With this exception, the bridge had been an unqualified success. To prevent shock to the traveller the car was suspended from its foot by links, and was received by a buffer-platform, backed by pneumatic cushions. Similar cushions were also provided to receive the traveller at the top. The clearance required in this case was 136 feet, which would render the sway with a rope suspension much greater than would be the case with the Author's design. The cost of the bridge had been approximately £23,000. In calculating the action of the wind, the live load had been considered as transferred entirely to the windward truss, and on the leeward side upper rails were provided for the uplift. The span was approximately but two-fifths of that of the Widnes and Runcorn bridge, and hence was more suitable for a simple span. He had, however, made preliminary estimates for a stiff braced structure of 1,000 feet span, with a clearance of 175 feet, and a stiff traveller with a live-load capacity of 150 tons, at an estimated cost of £120,000. Whether this estimate would be inconsistent with market-prices and labour-conditions in England he did not know, but it would seem that where such an excellent example of rigid construction of long span as the Forth bridge had been erected at so reasonable a cost, there should be no difficulty in putting up a 1,000-foot span in competition with the cable-suspension type, from the commercial standpoint; and a discussion by the Author of the relative costs of the suspension and the braced construction would be interesting and valuable. In the case of the Duluth bridge, a proposition had been submitted for a bridge of suspension type, with a capacity equal to one-third of the live-load of his design, at the same price, and naturally it had received but scant consideration, in view of the fact that its operation was guaranteed under wind-pressure but one-quarter of that for which the rigid type was designed.

The Author. The Author, in reply, observed that as Mr. Arnodin had designed all the transporter-bridges built before the one described in the Paper, his remarks were of special interest. The Author did not agree, however, with all Mr. Arnodin's statements, believing that several of them were not founded upon facts; he also failed to see the cause for regret that the improvements realized in other transporter-bridges had not been adopted. Before designing the

structure, the construction of existing bridges was carefully studied, and what appeared to be defects were avoided; this had led to the design of a bridge which was entirely different in many details from any that had been previously erected. With regard to the difference between the angles of the back-stay and the main cable, in the first place, the sharp angle of the back-stay was not due to design, but was governed entirely by the circumstances of the site, and could not possibly be avoided. In the next place, Mr. Arnodin's diagram (*Fig. 33*) certainly did not show the actual resultant pressure on the towers; for, by the adoption of saddle and rollers, the resultant pressure was made absolutely vertical; and the only possible use the diagram could be was to illustrate an elementary principle in mechanics. It was very necessary that the cable should be firmly gripped to the saddles, or there was undoubtedly danger of its slipping; and as the load increased during erection, care must be taken to follow up the diminution in the area of the cable due to the squeezing of the strands and of the individual wires, by screwing down the cable-grips as the work proceeded; for if by the carelessness of the engineer in charge this should be neglected, the result might be serious. With proper care there was not the slightest danger. When the cable and back-stays were continuous there was of course an excess of weight in the cables; but the extra cost of this excess in section was far preferable to, and cheaper than, multiplying the connections at the top of the towers. Mr. Arnodin's objection to the single cable, and his preference for a number of smaller ones, was founded upon the possible deterioration of the cable by percolation of water, and the impossibility of examining it; but surely, it was just as impossible to examine the inside of a cable 4 inches in diameter as it was that of a 12-inch cable, and the inspection was not assisted in the slightest degree by having a number of small cables. The great objection to the multitude of cables was the much larger surface which was exposed to the atmosphere; and as the atmosphere at Widnes was heavily charged with chemical fumes, it was very desirable that the surface of the cables should be exposed as little as possible. With ordinary care, the cables would last as long as, if not longer than, any part of the structure; and it was difficult to see, not only the advantage of the "principle of interchange" but the practicability of it; for it would be interesting to know how the various sections of the booms of the stiffening-girders, or the tower-legs could be interchanged. Even the cables would present many difficulties, on account of the many connections to the suspenders and to one another; in fact, in the Author's opinion, no such "interchanges" The Author.

The Author. would ever be made. The running of the trolley-wheels on one side of the flange of course produced eccentric loading; but certainly not with ill effects to the extent mentioned by Mr. Arnodin. Central loading was no doubt preferable, but the plan advocated by Mr. Arnodin and shown in his diagram (*Fig. 34*) had not been adopted because of the following grave defects:—As the wind-pressure was the most important factor in the consideration of the stresses, by adopting the proposed method there would be an additional distributed load of about 80 tons to be allowed for; which, for a span of 1,000 feet, was a serious item. The girder carrying the rails was entirely unsupported laterally, and the wind-pressure on the car, the reversal of the stresses, and the fatigue of the web next the angle-bar, would eventually produce serious results. The plan advocated necessitated the adoption of small wheels, which was a disadvantage. The four trolley-wheels on the same centre being rigidly connected with the trolley-framing, it would be next to impossible, with the varying deflection, for all the four wheels to bear alike on the rails; and the supposed evil of the rail on the outer flange would still exist. The supposed advantage of "interchange" of the wheels was imaginary; for a wheel could now be changed in less than 7 minutes. With reference to the question of propelling the car by cable or by direct traction, this had been carefully considered, and it had been decided that the latter had most advantages. With cable-traction, the two cables, of about 1,600 feet each, would have to be supported at short intervals, and would be a source of constant trouble in many ways, especially during a gale; the application of the brake to the car-trolley would present difficulties, and more power would be required to drive the car—not less power, as stated by Mr. Arnodin. Rope-traction had long since been abandoned for direct traction; and in travelling cranes, the conditions of which more nearly approached those of the transporter, rope-traction had not been adopted for many years and had been superseded by direct traction. With reference to the slipping of the driving-wheels, if the bridge were constructed according to the design of the Widnes bridge, with the rails having an ascending gradient at each end, there would be no fear of slipping, even during a gale, if the rails were not allowed to become greasy. The wear of the driving-wheels did not decrease their power of adhesion, as stated by Mr. Arnodin, for they adjusted themselves automatically to the changed diameter. The wearing of the wheels and of the rails was not such a serious matter as the wearing of the cables, pulleys, guides, adjusting-gear, etc., nor attended with such serious risks of breakdown: and

with rope-traction, if the ropes were to break, the car would be stranded. In reply to Mr. Colson, the strands forming the cable had been capable of adjustment at each end during erection, by means of the screws, and it had been easy to see when they had the correct versed sine and when they were in their place; then the temporary binding clips were put on. With individual wires this might be difficult; but with large strands it presented no further difficulty than that of the engineer being hung up in mid-air to make the inspection. The anchor-bars had been coated with tar-varnish in order to protect them not only during transit and erection but also after they were in position; for more than 12 months elapsed before they could be covered with the concrete, and in the meantime the pit was often full of water. Cement grout could be and was often used as a paint with advantage; but it required to be carefully put on, and the tar-varnish was inexpensive and easily applied. The concrete in the cylinders was practically hermetically sealed with the rock at the bottom and almost neat cement at the top, so that it might be considered to be perfectly water-tight. The erection of the cylinders had been carried out by a very experienced contractor, and he, knowing the condition of the site, had no doubt considered the plan he adopted to be the best; this matter had been left to the judgment of the contractors, and had not been specified by the engineers. The remarks of Mr. Duryea were interesting, but the Author could not agree with all his statements, nor with what he considered to be canons of good practice. As to Mr. Duryea's statement that it was unnecessary and unadvisable to cradle the cable, experience of the Widnes bridge had proved quite the contrary. Before the stiffening-girders met at the centre of the span during construction, a heavy gale sprang up; and but for the cradling of the cables, the results might have been serious, for the girders were not stayed in any way; as it was, the movement did not exceed 7 inches. If the versed sine of the cables had been one-eighth of the span instead of one-twelfth as suggested, the horizontal bracing of the girders to resist wind-pressure would have had to be considerably increased. Socketing the cables, if efficiently carried out as described in the Paper, was a very satisfactory device; and in the experiments made, the cable always broke outside the socket. With reference to the suggestion that stiffening-girders should be 56 feet deep instead of 19 feet, neither mathematically nor "without mathematics" could such a suggestion be satisfactorily upheld. Mr. Duryea's impression that the cost of the structure was unduly large, was not supported by the statements of many engineers and contractors of wide experience. Mr. Glover's remarks about the The Author.

The Author. construction of the cables formed a valuable addendum to those in the Paper. The question raised by Mr. Haynes about the comparative advantage of ropes and direct traction had been answered in the foregoing reply to Mr. Arnodin; his objection to the motive power being so high above the ground applied to both systems, but the driving-gear of the Widnes bridge was very accessible for any position of the car. The plan of the car had been made to suit local conditions, and was found to be very convenient; the broad car adopted on the Continent was necessary in order to obtain the required deck-area without projecting too much into the waterway; it was not such a convenient arrangement as that adopted at Widnes, which had two defined tracks for vehicles and ample space for foot-passengers. Mr. Henning had had considerable experience in the testing of cables and wires, and his opinions should have weight; but the Author was of opinion that further experiments on an extended scale, with large ropes, were necessary before the question of the relative advantages of parallel wires and twisted strands could be determined, and the result of the Author's experiments certainly coincided with those obtained at Watertown. If the stretching of the rope was "due to the rearrangement of the wires in the several strands" they would not have recovered when the load was taken off; and as the observations were taken near the working-load of the cables, after a load nearer the breaking-load had been applied, the modulus of elasticity obtained should be fairly accurate. Mr. Hildenbrand, who had had great experience in suspension-bridges, and had been associated with the design and construction of many of the largest well-known structures, asserted that there was no difficulty, as suggested by the Author, in arranging the parallel wires in their proper place and with the correct sag. If this were so, the best form of construction of the cable, for reasons mentioned in the Paper, was no doubt to have the individual wires laid parallel. In reply to Mr. Scholefield, the safe travel of the rollers was 8 inches, and as the travel of the rollers due to the extension and straightening of the back-stay during construction was about 10½ inches, the load was taken off the rollers by means of hydraulic jacks, and the rollers were set back during construction. The strands of the cables were temporarily clamped in position during erection at varying intervals, not always the exact 6 feet. The method of ascertaining whether all the suspension-rods had the correct tension was described in the Paper; and there was no reason why any one should be either under-screwed or over-screwed. The plan adopted by Mr. Turner in the Duluth bridge for carrying the trolley-wheels did not appear to be a wise one; for the wheels and the bearings, being inside a box boom, must be very

The Author.

difficult of access. The Author could not agree with Mr. Turner's ideas of the cumulative effects of wind; for no amount of stiffness of the structure would prevent them. The amount of vertical pressure on the booms of the girders due to a horizontal wind-pressure of 30 lbs. per square inch on the car, was about 8 tons, and the car worked perfectly well during a strong wind, the amperage of course being considerably increased. In the Duluth bridge, the longitudinal rigidity of the car-trolley was an undoubted mistake; for on account of the deflection of the girders with the travelling load, the end wheels of the trolley would carry at times most of the load; and it was probably more owing to this arrangement than to the defective "truck-bearings," that the trouble in working the car arose. It was absolutely necessary for the car-trolley frame to be flexible longitudinally.

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13 March, 1906.

Sir ALEXANDER RICHARDSON BINNIE, President, in the Chair.

The discussion on Mr. John J. Webster's Paper "The Widnes and Runcorn Transporter-Bridge" was continued and concluded.

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20 March, 1906.
Sir ALEXANDER RICHARDSON BINNIE, President,
in the Chair.

(*Paper No. 3565.*)

"The Outer Barrier, Hodbarrow Iron Mines, Millom, Cumberland."

By HARRY SHELFORD BIDWELL, M. Inst. C.E.

THE HODBARROW MINES.

THE County of Cumberland terminates towards the south in a promontory, which lies between the estuary of the River Duddon and the Irish Sea. This headland is known as Hodbarrow Point (Fig. 1, Plate 4), and is formed of carboniferous limestone, the highest part of the headland standing about 50 feet above sea-level.

Veins of red hæmatite are found in the limestone where it abuts on the shore, and the late William, Earl of Lonsdale, as Lord of the Manor, worked one of these veins at Towsey Hole (Fig. 1), and followed the vein for some distance into the hill; but as the workings proceeded the vein "nipped out," and the venture was ultimately abandoned. In 1855 the same Earl of Lonsdale granted to the founders of the Hodbarrow Mining Company the right to search for minerals over a large area in this locality, and boring-operations were begun, with the result that in 1856 one of the bore-holes passed through about 80 feet of solid hæmatite ore (X, Fig. 1, Plate 4), thus discovering the first deposit, which has now been almost entirely worked out. It was comparatively shallow, being covered by not more than 60 feet of limestone in any part; indeed, in one place the ore came almost to the surface, and it also rested on a limestone sole, the whole deposit being practically surrounded by limestone as indicated in Fig. 2, Plate 4, which is a diagrammatic section of the mines.

While this deposit was being worked, the company built workmen's cottages on the adjacent ore-mains, little knowing what lay

beneath the buildings; and it was an endeavour to supply these cottages with water that resulted in the discovery of the great body of ore which has since required so much outlay upon sea-defences for its successful extraction. A well had been sunk for some distance, but the supply of water being insufficient, in the year 1868 a bore-hole was put down at the bottom of the well, and this bore-hole pierced hæmatite ore at a depth of 180 feet below the surface. Boring being continued, it was proved that the ore-body extended over a very large area. Shafts were sunk without delay, and workings were begun in this main deposit in 1873, since which date an average annual output of about 450,000 tons has been maintained.

As will be seen from the section, the deposit has no limestone roof over the landward portion, but as the ore dips seaward a limestone roof begins and gradually thickens in this direction. It is probable that the ore was originally entirely covered by limestone, which has been partially denuded by the action of ice, and that the boulder clay took its place when the glacial period passed away. Above the boulder clay lay irregular beds of sand, gravel and clay, apparently deposited by the sea, as marine shells were found therein while shafts were being sunk through these beds. Difficulties arose occasionally through rushes of sand and water from these overlying strata into the workings of the mine, and very powerful pumping-machinery had to be provided to deal therewith. Another danger arose from the fact that the sea was encroaching upon the low sand-banks which formed the sea-margin, and with a view to stop this encroachment a timber revetment was erected in 1882-4.

Timber Revetment.—This first attempt at sea-defence was formed, at the site indicated in Fig. 1, of 6-inch timber sheeting driven 6 to 8 feet into the ground, with walings in front, and main piles of pitch-pine 12 inches square, driven 20 feet. The main piles were tied with iron rods to back piles driven at a distance of 60 feet behind them. This structure, however, was breached by the sea, in November, 1884, during a heavy gale and exceptionally high tide.

The company's lease of the minerals extended only to high-water mark of ordinary spring-tides on the south, and inasmuch as the surface caved in when the ore was extracted, it was absolutely impossible to work out all the ore up to that limit, a barrier of unworked ground 360 feet wide having to be left to protect the mines from the sea. Drifts in solid ore were, however, carried up to and along the seaward boundary, proving beyond doubt that a considerable field of ore existed outside the area then leased by the company. The lessor, however, was not at that time

disposed to grant an extension of the area so leased, although the foreshore was vested in him; but when it became apparent that unless some more efficient provision was made for sea-defence there was considerable risk of the mines being flooded, consent was given in 1886 for the construction of the first sea-wall (Fig. 1, Plate 4), as designed by the late Sir John Coode, Past-President Inst. C.E., which was completed in 1890. This wall is a rigid structure of concrete, backed by a clay embankment, and rendered water-tight by a wall and trench of puddled clay immediately behind the concrete wall, keyed into the natural clay bed beneath. The site of this work was so arranged that while it afforded an absolutely water-tight barrier, it also enabled the company to extract all the ore up to the then existing boundary. The lessor at the same time granted the company permission to put down bore-holes on the foreshore for the purpose of ascertaining the extent of the ore-ground seaward, and under these powers boring-operations were continued for some years under great difficulties, the gear and staging being frequently carried away bodily by the sea, which in south-westerly gales is very heavy on this part of the coast. In spite of this, however, the bore-holes, which were arranged in sectional lines, were successfully carried out, and an extent of ore-ground large enough to make a further enclosure desirable was proved.

In 1898 a bed of quicksand was tapped in the mine, which established a connection between the sea and the underground workings, a cavity being formed on the foreshore outside the sea-wall; and a heavy rush of tidal water into the mine took place, passing many fathoms below the foundations of the sea-wall. This inflow was checked by filling up the cavity on the foreshore with furze, brushwood and clay, but not before the sea-wall had begun to show signs of distress through deflection caused by the removal, by the current, of material from beneath it. The clay embankment behind the wall then subsided about 5 feet, and this subsidence had the effect of shutting off the connection with the sea, and the influx of tidal water shortly afterwards ceased. Thereafter, upon the advice of Mr. William Matthews, Vice-President Inst. C.E., the damaged portion of the sea-wall was protected from heavy strokes of the sea by a wave-breaker of pell-mell blocks of concrete, each 20 tons in weight; the subsided embankment behind it was levelled up, raised in height, and added to in width, thus giving additional weight to aid in shutting off the leakage into the mine; while the cavity on the foreshore was surrounded with sheet-piling, and within the enclosure clay was deposited, which was covered with a heavy layer of limestone.

This accident had the effect of hastening the negotiations for the erection of a further protective structure, and the lessor having granted to the company a new lease of the mines, including the foreshore minerals, the work of designing an outer barrier to exclude the sea from this extended area was entrusted to Messrs. Coode, Son and Matthews in the same year. The contract for the work was let to Messrs. John Aird & Co. in 1899, and the construction was begun early in 1900

General Description of the Outer Barrier Works.

The area reclaimed by the construction of the Outer Barrier is 170 acres. That the whole of this ground is not ore-bearing, though a very large portion of it is so, will be seen from Fig. 1, Plate 4, whereon the approximate edges of the ore-deposit are shown. It was necessary, however, to include a certain portion of barren ground, more especially upon the west side, where it was considered advisable that the Barrier should be located a considerable distance outside the treacherous ground in which the subsidence before alluded to had taken place.

In the design of the Barrier the primary objects kept in view were, first, that the structure should be flexible instead of, as in the case of the first sea-wall, rigid, in order that, should subsidence take place beneath it at any time, the Barrier itself should also subside, and thus close any cavity that might be formed, when, by adding material to the superstructure, its efficiency as a protection to the mines would be maintained; and, secondly, that it should be of great weight, in order to compress the upper stratum of sand on the foreshore and thus aid in preventing percolation beneath it, and also to secure its subsiding if necessary. A further consideration to which prominence was given was that, as far as possible, material obtainable in close proximity to the work should be utilized in its construction.

As will be seen from Fig. 3, Plate 4, the Barrier has the form of an arc, the east end springing from Hodbarrow Point, while the west end is situated near the mouth of the Haverigg River. It is 6,870 feet in length (Fig. 4), and consists primarily of a main outer bank of limestone, an inner bank which it was originally intended to construct of iron slag, but which, for reasons to be stated hereafter, was largely composed also of limestone, and a filling of clay between these two banks (Figs. 5). These materials fulfil the condition mentioned in the preceding paragraph, there being, as already stated, large deposits of limestone overlying

the ore of the Hodbarrow Mines, while clay was obtainable in large quantities upon the property of the Mining Company, and slag was readily procurable from the adjacent works of the Millom and Askam Hematite Iron Company.

As a protection against heavy waves the outer limestone bank is provided, for 1,600 feet from the shore at each end, with a coating of large lumps of rough limestone weighing up to 15 tons. Seaward of this the protective coating is formed of 25-ton concrete blocks deposited irregularly.

For the prevention of percolation of water beneath or through the Barrier, the weight of the structure and the filling of clay alluded to above were not alone relied upon. Beneath the surface of the foreshore, under the centre of the clay embankment, a water-tight cut-off is provided, and for this three methods were adopted: first, where the natural clay bed was near the surface the cut-off was formed by a trench of puddled clay, carried down and well keyed into the clay bed; secondly, where the clay bed existed, but was at too great a depth to be reached by means of trenching, pitch-pine sheet-piling, grooved and tongued, was driven down into the clay; and, thirdly, where no clay was found, or where the bed was so irregular as to be unreliable, steel sheet-piling, of lengths varying according to circumstances, was driven. In order to guard against the passage of water through the Barrier, a wall of puddled clay is constructed in the heart of the clay embankment, immediately above the puddled trench or the piling, as the case may be. It is brought up to 5 feet above high water of ordinary spring-tides, which is 6 inches above the highest recorded tide, a level that has been reached only once, and approached but seldom, in a large number of years.

For the purpose of discharging water which might accumulate in the reclaimed area, four sluice-ways are provided, having pen-stocks worked from the road-level of the Barrier and tide-flaps at the seaward ends.

The work of construction was started at the west or Haverigg end, but operations were shortly afterwards begun at the east end, and were thereafter carried on at both ends simultaneously.

The order of procedure was as follows:—The formation of the water-tight cut-off below foreshore-level was first begun, followed by the main limestone bank and the inner bank; the latter was kept slightly behind the former with a view to shelter, while the filling for the clay portion of the embankment was deposited after such an interval as would permit of the formation of the lower portion of the puddle wall in the manner hereafter described.

Borings.—In order to determine the method of construction of the water-tight cut-off below foreshore-level, borings were made at intervals of approximately 600 feet along the proposed line of the work, and opposite each of these a second boring was made seaward of the line of work; while at the west end, opposite to the spot where the inrush of water had occurred in 1898, a third, and in one case a fourth, boring was made on each line. These borings indicated the existence of a good clay bed along the greater part of the line of work, at a depth below the surface varying between 16 and 20 feet at the west end, between 7 and 15 feet along the central portion, and between 3 and 11 feet at the east end; there being two intermediate portions where either no clay was found, or the bed was only thin. From the data thus obtained it was decided to form the cut-off in the following manner, the distances being measured from the west end of the work:—

Puddled trench	0 feet to 350 feet.
Pitch-pine sheet-piling	350 ,, ,, 1,200 ,,
Steel sheet-piling, 34 feet 6 inches in length	1,200 ,, ,, 2,400 ,,
Pitch-pine sheet-piling	2,400 ,, ,, 2,940 ,,
Puddled trench	2,940 ,, ,, 4,133 ,,
Site of sluice-culverts; concrete foundations carried down to clay . . .	4,133 ,, ,, 4,166 ,, 6 inches
Steel sheet-piling, 31 feet 6 inches in length	4,166 ,, 6 inches to 4,662 feet 6 inches.
Puddled trench (east end)	4,662 feet 6 inches to 6,870 feet.

Puddled Trench.—A beginning was accordingly made with the trench at the west end, but the work of excavating the trench had not proceeded very far when difficulties were encountered. It had been anticipated from the results of the nearest borings that the clay bed would be found at about 12 feet below the surface, but a length of 80 feet of trench was opened out and carried to a depth of 15 to 20 feet without coming to clay. Additional borings were then made between the shore and the first of the original borings, and these revealed the fact that the clay, instead of rising towards the shore, was apparently horizontal for about 350 feet and then dipped downwards. This being so, it was decided that the attempt to carry out this portion of the work in puddled trench should be abandoned, and that pitch-pine sheet-piling should be substituted.

At the east end the puddled trench was carried out for a length of 1,040 feet from the shore. Of this length the first 200 feet was carried down to rock, into which it was keyed to a depth of 2 feet, the recess being excavated by blasting with light charges of dynamite to receive the footing of the puddle. Beyond

this point, for a distance of 180 feet, as will be seen from the longitudinal section (Fig. 4, Plate 4), clay appeared on the surface; but on excavating in this clay to carry the puddle into it to the specified depth of 1 foot 6 inches it was found not only that the clay was of an unsatisfactory character but also that it was underlaid by a water-bearing stratum of gravel. The trench was therefore carried down through this water-bearing stratum and keyed into the rock or solid clay which was found beneath. From this point outwards the trench was taken down through sand and gravel to the clay bed, which was reached at a depth varying between 2 feet and 13 feet.

The trench was generally 7 feet wide, and where the depth necessitated the use of an inner setting of the timber runners or poling-boards by means of which it was sunk, this inner setting had a minimum width of 6 feet. The bottom of the trench, in the clay, was 4 feet wide, being tapered to this width in the depth of 1 foot 6 inches to which it was carried in the clay. The work was undertaken in sections whose length varied according to the depth of the trench: an average length in a trench of a single setting was 20 feet, while in deeper trenches the length was about 10 feet. This work was of course carried on only while the tide was out, the length of the working-period being at first as much as 7½ hours, which naturally became materially reduced as the work progressed seaward.

As soon as the tide had receded sufficiently to permit of starting work, the water remaining in the portion of trench under construction was pumped out, one 6-inch and two 8-inch centrifugal pumps being used for this purpose. When the trench was dry enough, excavation was proceeded with, the excavated material being filled into tubs which were lifted from the trench by a crane and tipped landward of the site of the inner bank at a sufficient distance to prevent the spoil from being carried back into the work. When the clay bed had been reached, and the footing had been cut into it as previously described, the bottom was carefully cleaned and the puddled clay was deposited. The clay for puddle was obtained principally from fields lying near the west end of the work, where the clay was of especially good quality. Here were two steam pug-mills, whence the puddle was taken to the work. The puddle was deposited in layers of about 1 foot, well trodden down, especial care being taken in forming the junction between the puddle and the natural clay bed. Where possible, the side timbering of the trenches was withdrawn as the puddle was brought up, the latter being then well rammed against the sides of the excavation. In some cases, however, the nature of the surrounding ground rendered it necessary that this timbering should be left in; indeed,

for a considerable length of the work it was necessary to drive permanent 6-inch sheeting on either side of the trench.

When the work had proceeded for some time, the clay bed at this east end began to be very irregular, sometimes dipping almost vertically both in the line of and transversely to the work. Thus, in the width of the trench, 7 feet, there would be a difference of level of as much as 4 feet in the surface of the clay. It was therefore seen that, even if conditions became no worse, it was very doubtful whether a satisfactory cut-off could be made by the proposed method, and early in 1902 a series of borings was commenced at close intervals between the then existing end of the puddled trench and the point at which it had been proposed to terminate the 31-foot 6-inch steel piling. These borings were made on the line of the cut-off and were therefore 25 feet landward of the centre-line of the Barrier, on which line the original borings had been taken. They demonstrated the correctness of the original borings, but disclosed great irregularities in the clay bed between them. The original borings at 5,066 feet and 5,692 feet had chanced to be situated on the summit of ridges in the clay, with deep valleys between them and also westward of the boring at 5,066 feet. How irregular the surface of the clay was will be seen on reference to Fig. 4, Plate 4. In one instance there was a difference in level of 26 feet in 25 feet distance, while between two points 100 feet apart the difference was no less than 43 feet.

Having regard to the information disclosed by these borings, it was decided to discard the puddled trench as the means of forming the cut-off on this section of the work, and to substitute for it steel sheet-piling, 31 feet 6 inches in length where the surface of the clay was reached by this depth, and 34 feet 6 inches in length across the two deep valleys in the clay before mentioned. It was now considered advisable that the additional borings should be continued along the remaining section of the work where it had been intended to form the cut-off by puddled trench, and this was accordingly done. Here no very great variations were found, the surface of the clay being in no case deeper than 3 to 4 feet below the line assumed from the original borings at 3,148 feet and 3,791 feet. Nevertheless, it was apprehended that the puddled trench might not prove satisfactory here, in view of the curious lateral irregularities which had previously been met with, and it was determined that here also it should be superseded by steel piling, though it was not considered necessary that the latter should be as deep as that used for the other sections of the work, 25 feet 6 inches being deemed sufficient.

Timber Sheet-piling.—The piles for this work were whole timbers, not less than 12 inches by 12 inches, grooved and tongued. For the first 150 feet of the work the piles were 18 feet in length, and thence onwards they varied between 23 feet and 27 feet, according to the depth of the clay, into which they were to be driven 4 feet. The piling was driven in bays of 10 feet each, that is to say, the gauge-piles were spaced 10 feet apart between centres, and they were driven 2 feet 6 inches deeper than the sheet-piles. A series of gauge-piles was first driven by one piling-engine: a second engine then followed, and drove the eight sheet-piles in each bay, also completing the bay by driving the central key-pile. The grooves in the piles were $3\frac{3}{4}$ inches wide and $1\frac{7}{8}$ inches deep, and the tongues $3\frac{1}{2}$ inches by $3\frac{1}{2}$ inches, the latter being secured to the piles by 10-inch spikes driven 18 inches apart. When driven to the requisite depth, the heads of these piles stood 3 feet above the ground-level, and at 1 foot from the top a waling 12 inches by 6 inches was fixed on the landward side of the sheeting, being secured to each pile by a $1\frac{1}{8}$-inch bolt.

The engines used for driving this piling were of the ordinary type, the monkeys weighing 22 cwt. The drop generally allowed was 4 to 5 feet, and the rate of driving varied between 1 inch at the beginning and $\frac{1}{8}$ inch at the end, per blow, being affected also by the nature of the stratum through which the pile was being driven. In the case of the key-piles the driving was usually somewhat slower.

Where obstructions were met with, a shaft was sunk and the obstruction was removed, or if necessary the pile was drawn and started afresh. In two places neither of these courses was feasible, and here a boxing of piles was driven on the seaward side of the cut-off to enclose the defective part, the enclosure being subsequently filled with puddled clay.

During the progress of this piling it was possible to ascertain fairly accurately, from the changes in the speed of driving, the levels at which a pile entered the different strata: this was specially noticeable on the pile getting through the bed of gravel and entering the clay, when the freedom in driving increased in a marked degree. A considerable length of the piling had been driven without indicating any notable deviation in the level of the clay bed as shown by the original borings; but towards the end of 1900 there appeared to be indications that the clay was being met with at a higher level than had been anticipated, and on borings being put down between 900 feet and the next boring at 1,154 feet a ridge was found to exist here. It was not considered advisable, however, that the piling should be curtailed in length

here, and it was accordingly driven to the full depth originally proposed (Fig. 4, Plate 4).

The piles having been driven, as each two or three bays were completed the sand on either side of the sheeting, between the outer and inner stone banks, was excavated to form a basin-shaped hollow 4 feet deep at the piling and 34 feet in width where the distance between the two banks would admit of this. The hollow, having been cleared of weeds, mud, etc., was filled with puddled clay, thus providing a flap which should prevent the creeping of water at the upper portion of the sheeting and form the foundation of the puddle wall.

When the first length of 1,200 feet of this timber sheet-piling had been driven, the succeeding section of steel piling was proceeded with and completed, and a beginning was made with the second series of timber piling.

The ground through which the latter portion of the steel piling had been driven had proved exceedingly hard, and conditions became no more favourable when the timber piling was proceeded with. Pile after pile refused to be driven, and showed obvious signs of being crippled. When attempts were made to draw these piles, some parted, only the upper portion being drawn; others came up without their shoes, and with their ends burred into fibre. As these occurrences happened at the same time as the difficulties in the puddle trench, the borings in connection therewith were continued, in order to determine the exact nature of the ground over the site of this proposed length of timber piling. These borings, like those in the adjoining stretch of ground, did not reveal any great irregularities in the clay bed; but it was considered impossible, in view of the experience just detailed, to ensure that timber piling would successfully penetrate the bed of hard gravel and stones and make a satisfactory junction with the clay; and it was decided that this piling should, like the puddled trench, be superseded by steel piling, to be 25 feet 6 inches long and therefore going some 5 feet deeper than the timber piling.

The whole remaining portion of the cut-off was therefore executed in steel piling, forming, with the section already driven, a continuous diaphragm $\frac{7}{8}$ mile in length.

Steel Sheet-Piling.

Original Design.—The first design for the steel piling consisted of a system of gauge-piles, spaced 11 feet apart between centres, with four sheet-piles and a central key-pile between. The gauge-piles

consisted of rolled steel H-shaped joists, 9 inches by 7 inches by $\frac{3}{4}$ inch, having jaws formed by 3-inch by 3-inch by $\frac{1}{2}$-inch angle-bars riveted to the flanges. The flanges of the joists were cut away at the bottom to form a point. The sheet-piles were of $\frac{3}{4}$-inch steel plate, 2 feet wide, having on one side 3-inch by $\frac{3}{4}$-inch stop-bars so placed as to butt against the jaws of the adjoining pile and leave $\frac{1}{8}$ inch play in driving, and on the other side bars forming jaws to receive the next pile. On the pile adjoining the gauge-pile these bars were 7 inches by $\frac{3}{4}$ inch, making a groove $3\frac{1}{8}$ inches deep, while the next pile had bars 10 inches by $\frac{3}{4}$ inch, giving a $6\frac{1}{8}$-inch groove. These piles had cast-steel shoes, so bevelled as to cause each pile to draw close to its previously-driven neighbour. The key-piles consisted of $\frac{3}{4}$-inch steel plate, 1 foot $10\frac{1}{2}$ inches wide, stiffened by a 4-inch by 3-inch by $\frac{1}{2}$-inch T-bar on the front and having a cast-steel shoe. The width of the key-pile allowed 2 inches play on either side between it and the adjoining sheet-piles. The gauge-piles were made in one length, but the sheet- and key-piles were made in halves, the upper and lower lengths being secured together, after the driving of the latter, by cover-plates 1 foot 10 inches long over the plate of the pile, and 2 feet 6 inches long over the stop-bars and groove-bars in the case of the sheet-piles, and by a channel-bar $9\frac{1}{2}$ inches by $3\frac{1}{2}$ inches by $\frac{1}{2}$ inch and 4 feet long riveted through both plate and T-bar, in the case of the key-pile. Cast-steel driving-heads were provided, which were suitable for driving either top or bottom lengths of the piles.

First Trial Bay.—It had been provided in the contract that, before deciding finally that the steel piling should be of this form, material for one bay of the 31-foot 6-inch piling should be prepared, consisting of two gauge-piles, four ordinary sheet-piles, and one key-pile; and that this should be driven near the site of the works, as an experimental bay. A spot was accordingly selected near the west end of the work, and, it having been ascertained by sinking a pit that the ground was similar to that which it was anticipated would be met with in the line of the work, driving-operations were begun. The two gauge-piles were first driven, followed by the two piles adjoining them. Next the lower halves of the two remaining sheet-piles were driven, and the lower half of the key-pile was then started. Up to this stage the driving had proceeded in a satisfactory manner, but when the point of the key-pile was a few feet below the surface, driving became exceedingly hard and eventually progress practically ceased. It being evident that some obstruction had been encountered, a pit was sunk to the foot of the pile, when it was found that the point had struck a boulder, and that, having

thus been started out of its true course, it had gradually turned up until at 10 feet from the surface of the ground it was at right angles to the upper portion of the pile, the upturned portion being that below the end of the stiffening T-bar. In turning up it had burst open the groove-plates of the sheet-piles, the grooves of which were found to be filled with pebbles and sand compressed to the consistency of concrete.

The three half-piles were drawn, and the grooves of the sheet-piles were repaired; the bent point of the key-pile was removed; and a new driving-point was formed by cutting away the corners of the plate and riveting additional T-bars, 2 feet 3 inches long, on either side of the central bar on the front of the pile, and three similar bars on the back. Driving was then recommenced, and the piles were driven to the intended level. An examination-pit was sunk at the back of the piling, when it was found that, although the piles were in a satisfactory condition down to the joints, they were not so below that level. The bottom 3 feet of one of the piles adjoining the key-pile was turned completely upwards in the form of a hook; the plate was much torn; and the groove-plates were stripped from this portion. The corresponding pile on the opposite side of the key-pile had left the groove of its neighbouring pile, and was much twisted; while the key-pile had also left the grooves of the sheet-piles, being 1 foot 6 inches away from one of them at the foot[1] (*Figs. 6*).

Revised Design (Figs. 7, Plate 5).—It being thus evident that this arrangement of piling would not be satisfactory, further designs were prepared, and it was ultimately determined to make trial of a system of alternate gauge- and sheet-piles. For this the gauge-piles remained unaltered, and the sheet-piles consisted as before of $\frac{3}{4}$-inch plate, 2 feet wide, but were stiffened by having central ribs formed of 5-inch by 3-inch by $\frac{1}{2}$-inch T-bars riveted on back and front. The cast-steel shoes were done away with, and the driving-point was formed by T-bars, 3 feet 6 inches long, riveted on each side of the central tee. The point of the pile was shaped to an angle of 30 degrees and the edges were cut so as to form horns to aid in clearing the grooves of the gauge-piles of sand and pebbles. As time was of importance, the material of the first trial bay was adapted for the second experiment, the two less-injured sheet-piles being altered to the new design, and a third gauge-pile being extemporized from a 12-inch by 6-inch by $\frac{1}{2}$-inch joist which was on the spot.

Second Trial Bay.—These three gauge-piles were then driven, a

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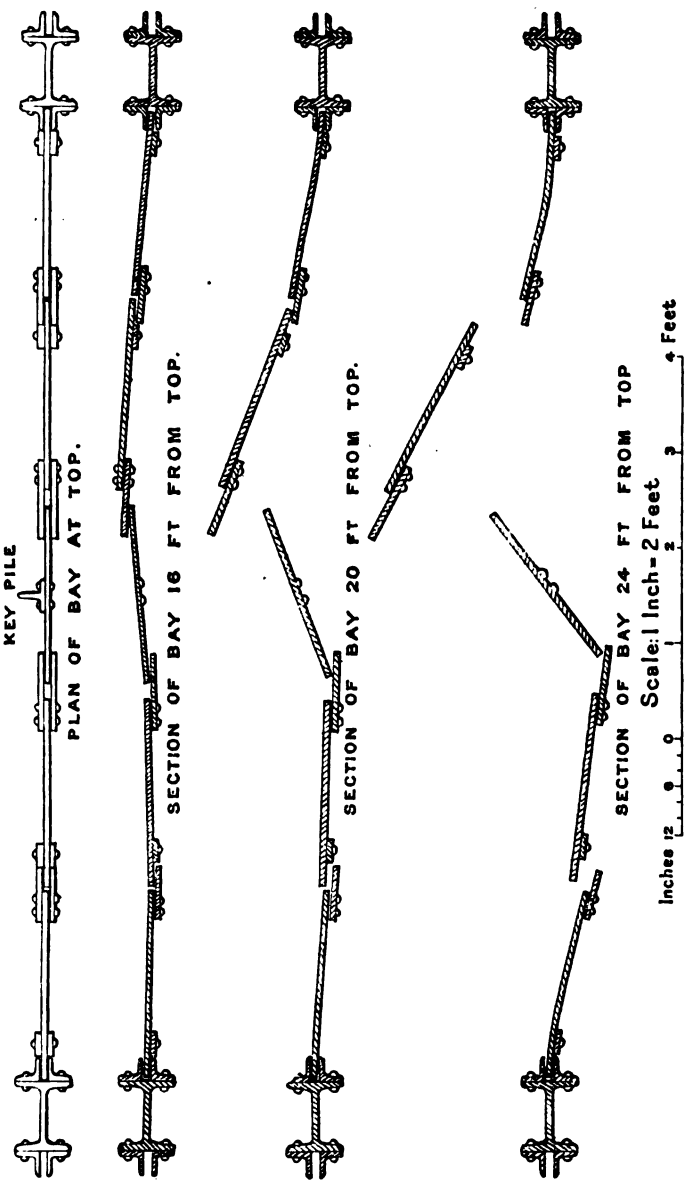
[1] The plates, etc., on the front of the piles are not shown in *Figs. 6*, as their condition could not be ascertained.

clearance of 2 feet 1 inch being allowed between them. The lower

Figs. 6.

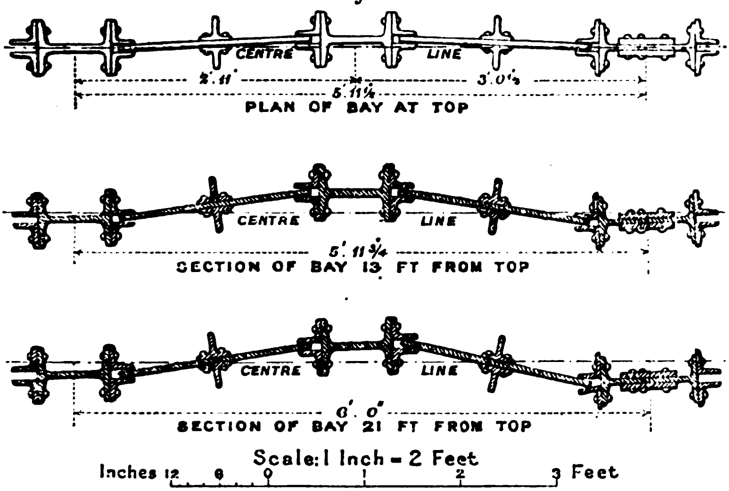


FIRST TRIAL BAY OF STEEL SHEET-PILING.

halves of the two sheet-piles were next driven, after which the

upper halves were riveted on and the driving was completed. As before, a pit was sunk, and the piles were duly examined, when it was found that a perfect cut-off had been formed (*Figs. 8*), the sheet-piling being continuously in the grooves of the gauge-piles for a depth of 29 feet from the top, below which point it was not practicable to make an examination, owing to land-water rising in the pit. There was no indication, however, of any tendency to irregularity. During the rising of the tide on the day of the examination, the water in the pit was tested at intervals with a salinometer for the purpose of ascertaining whether percolation of sea-water was taking place. No such percolation could be detected, although the bottom

Figs. 8.



Second Trial Bay of Steel Sheet-Piling.

of the pit was 17 feet below the level of high water of ordinary spring-tides.

The second arrangement having thus been found to be satisfactory, it was decided that the work should be carried out on these lines; though only a portion of the material required was procured, lest any further modification should subsequently be found necessary. The work of driving the first series of steel piles was begun in February, 1901.

Arrangements for Driving Piles.—Short timber piles were first driven, to carry the platform on which the engines for driving the steel piling were to work, and for affixing the arrangement for keeping the line of piling true. These piles were spaced 10 feet apart between centres in the direction of the line of works. The

two front rows were respectively 1 foot 4 inches in front of and behind the line of the sheeting, and to them were fixed 12-inch by 12-inch walings, the space left between these being thus 8 inches. These walings were placed so that their upper side was 1 foot 9 inches below the finished level of the top of the steel piling. Between these walings, and secured to them by coach-screws, were fixed distance-blocks, the space between these being exactly that occupied by a gauge-pile, namely 1 foot 3 inches, while their width allowed a clearance of 2 feet 1⅛ inch between the flanges of the gauge-piles, the sheet-piles being, as stated, 2 feet 1 inch wide. These blocks had rubbing-pieces of ¼-inch iron plate at each side, to take the edges of the angle-jaws of the gauge-pile. A considerable length of staging and guides was prepared in this way, so that work could be started at several places at once. As each gauge-piling engine completed the driving of a sufficient number of gauge-piles a sheet-piling engine followed it up. The lower lengths of several sheet-piles were first driven; their upper lengths were then riveted on, and the driving was completed.

Behind the platform on which the piling-engines stood was a line of railway, along which were brought the boilers for driving the steam-winches of the engines. These were 6-HP. and 8-HP. vertical boilers mounted on railway-wagons, and were connected with the winches by means of flexible armoured hose. They were, of course, brought down when the tide had receded sufficiently, and drawn up on shore again at the close of the working-period.

The driving-gear for the gauge-piles comprised a 30-cwt. monkey, 22-cwt. dolly and 3¼-cwt. driving-head or cap, all made of Hadfield toughened steel. For the sheet-piles a similar monkey was used, but the dolly and cap weighed 19½ cwt. and 4¾ cwt. respectively. On the top of the driving-cap, between it and the dolly, was placed a pad of compressed fibre, 1⅛ inch thick, to diminish jarring; while in the case of the sheet-pile gear a strip of copper was also beaten into the groove in the cap which fitted over the head of the pile, for the same purpose. At an early stage of the work difficulties were frequently experienced with the attachments of this gear, the monkey-eyes, gauge-bolts, and cotters becoming crystalline after a few hours' work and breaking. Various qualities of steel and other metals and alloys were tried for these parts of the gear, but eventually a return was made to commercial iron of good quality, rubber pads being inserted between the parts to reduce the jarring; and this increased considerably the life of the attachments, and of the monkeys, etc.

In driving the gauge-piles, any tendency shown by the pile to run

outwards from the machine or in towards it was counteracted by packing up the engine at either the back or the front, and so giving it an inclination outwards or inwards, which had the desired effect in bringing the pile back to the plumb; while to check any tendency to run sideways a winch with blocks and tackle was provided, by which a strain was kept on the head of the pile in the necessary direction.

Use of Water-jet.—As already mentioned, some very hard ground was met with in driving the steel piles, and as an aid in driving through this ground a water-jet was used. For this double-ram pumps were employed, of 5½-inch and 6-inch diameter, worked by 6-HP. and 8-HP. vertical boilers; or at times two pumps were worked by a 10-HP. boiler. The water was delivered from the pumps by a 5-inch pipe, diminishing to 3 inches and again to 1½ inch in diameter; from this a 1¼-inch vertical pipe led to the jet-pipes, of which there were two, springing from a T-joint and coming down in front of and behind the pile respectively. These down pipes were tapered to a point ¾ inch in diameter. The pipes were kept working up and down by hand by means of a block and tackle, the points being kept about level with or slightly below the point of the pile. It had been feared that the use of these jets might unduly disturb the surrounding ground, leaving permanent ill-effects; but experiments made before regular adoption of the system proved that such was not the case. After one tide had flowed over the ground it became fairly hard again, and after three or four tides it was found to have returned to its normal consistency. The points of these jet-pipes were not allowed, save in exceptional cases, to be put down lower than 8 feet above the finished level of the point of the pile. The jet was used in driving the sheet-piles only, as the gauge-piles could not have been satisfactorily guided had it been used for them.

Fig. 10, Plate 5, shows the acceleration in speed of driving attained by the use of the water-jet. The fall of the monkey was 3 feet, as it was found that, having due regard to wear and tear on plant, especially on the monkeys, dollies and driving-caps, this should be the maximum fall allowed.

In certain exceptional cases, periods of very hard driving of sheet-piles were experienced. Some of these were caused by meeting with obstructions, probably small boulders, in the gravel bed, as they usually occurred during the passage of the point of the pile through this stratum. In one case a pile which had progressed at about the ordinary speed until 7 feet from the finished depth, met with such an obstruction at this point: the driving became gradually harder, until at 3 feet from the finish one hundred and sixty blows of the

monkey were required to drive 1 inch, the pile apparently carrying the obstruction down with it. Then the obstruction appeared to be gradually pushed aside, as the driving became easier, although still hard, the driving for the last 2 feet being at the rate of forty-eight blows per inch. Occasional hard driving of the sheet-piles also occurred, where the gauge-piles had not been driven quite truly and the sheet-pile ground against either the face of the flange, or the sides of the jaws, of the gauge-pile. In one instance the sheet-pile evidently cut into the flange of the gauge-pile, as it began to carry the latter down with it. An attempt was made to draw the sheet-pile, but without success, and eventually the gauge-pile was driven down by means of a special dolly until the sheet-pile was released, when the latter was driven to finished depth at about the normal rate of driving. The gap which was necessarily left between the two adjoining sheet-piles by this extra driving of the gauge-pile was closed by a plate, secured to each sheet-pile by fishplates.

With the gauge-piles abnormally hard driving was not very frequently experienced, Figs. 9, Plate 5, being representative of the general rate of driving. Occasionally, however, a pile would be diverted from its course to too great an extent to permit of the deviation being rectified by the manipulation of the piling-engine or side-winch to which reference has been made; such piles were drawn by means of hydraulic jacks and repitched, the second attempt to drive them usually proving successful. Where, however, it was evident that an obstruction existed, either to gauge-pile or to sheet-pile, which would entirely prevent the pile from being driven, a caisson was sunk and the obstacle was removed. On some occasions it was necessary to sink both in front of and behind the line of piling, and two caissons were therefore provided. These caissons were 6 feet wide by 5 feet across, and were formed of steel plate stiffened by T-bars. They were made in sections, each 3 feet deep, having flanges formed of angle-bars, so that the successive sections could be bolted together as the sinking proceeded. The bottom length was provided with a cutting edge. By this means obstacles were removed on several occasions. The largest of these were met with in the clay bed and consisted of granite boulders.

When the piles had been driven until their heads stood 2 feet 3 inches above ground-level the adjacent ground was excavated and filled with puddled clay in the manner already described in the case of the timber sheet-piling. By the time that about 400 feet of the piling had been driven it was felt with confidence that the design

was satisfactory, and the remainder of the piling was accordingly all of the type described. The total length of cut-off formed by steel piling was 4,636 feet, the various lengths used being :—

34-foot 6-inch piles	. . . .	1,720 feet.
31-foot 6-inch ,,	. . . .	1,258 ,,
25-foot 6-inch	. . . .	1,658 ,,
	Total	4,636 ,,

Embankment.

Turning now to the structure of the Barrier above the foreshore-level, the top of the main outer limestone bank is 15 feet above high-water level of ordinary spring-tides, this bank having an extreme height of 40 feet above the foreshore. The top of the inner bank is at high-water level, while that of the clay hearting is at an upward incline of 1 in 40 landward from the inner edge of the main limestone bank, its landward side being sloped down to meet the outer edge of the lower stone bank (Figs. 5, Plate 4).

The main limestone bank is 10 feet wide at the top, exclusive of the width of the protective coating of larger material. It has slopes of 1 to 1 on the landward and 1½ to 1 on the seaward side. The bank was not in the first instance formed to its full height, but was kept 5 to 6 feet below this, in order to provide sufficient width for the requisite number of construction-roads on the top: the upper layer was deposited later, as the progress of the work permitted. The stone was obtained from quarries adjacent to the east end of the work, and consisted of large and small material mingled together, a preponderance of the larger being used as far as possible for the sides of the banks, especially the seaward side.

The practice in tipping this bank was to keep two tip-heads working, with end-tip wagons, following up at intervals with side-tipped material. For tipping the larger material in the main bank, and also the protective coating of large limestone lumps, special wagons were built, the end-tipping wagons to carry 12 tons and the side-tipping 15 tons. The former had wheels 3 feet in diameter, this increase in size over that of the wheels of ordinary tip-wagons being found advantageous, inasmuch as the wagons ran much more easily, and also were more readily spragged.

Quarrying.—The method of quarrying was so arranged that the bulk of the material should be of large size, and the blasting was carried out so as to avoid unduly shattering the rock, the object

being, of course, to minimize the likelihood of disturbance of the stone by the sea after it had been deposited in the bank. Most of the holes for blasting were drilled by Hirnant steam-drills, of which twelve were employed. Two or more of these were worked from one vertical boiler, there being two 8-HP. and two 10-HP. boilers in use for this purpose. The rate of work accomplished by these drills in a hole beginning with a diameter of 3 inches and diminishing to 1½ inch averaged 2¼ feet per hour. The depth of the holes was dependent, of course, upon the position of the beds in the rock, and varied between 4½ and 25 feet.

The system of blasting was ordinarily by "chambering," for which purpose a gelignite charge of 1½ to 3 lbs. was first used; this was followed by a charge of blasting-powder of 8 to 20 lbs., and then the final charge, also of powder, which varied between 130 and 270 lbs. Firing was done by the Bickford fuse. By these means masses of stone weighing up to 15 tons were dislodged, which were used for the protective coating of the bank. This coating was 15 feet in width and was deposited mainly from side-tipping wagons, the lumps being, where necessary, hauled down into place with chains from hand-winches on the shore, the finishing being done by depositing the stones with steam-cranes. An ordinary day's output from the quarry to the embankment when work was in full swing was 900 cubic yards.

Concrete Wave-breaker Blocks.—The covering of 25-ton concrete blocks, with which the central portion of the Barrier is faced, is 25 feet in width, and consists generally of successive tiers of two blocks, each placed at right angles to the line of the work, so that their ends are presented to the stroke of the sea. These blocks are not placed flat on their beds, but are deposited as irregularly as was consistent with avoidance of an undue proportion of interstices in the work, and with securing a sufficient weight of block-work. The blocks measured 11 feet by 6 feet by 6 feet, and the concrete was composed of 6 parts of sand and shingle to 1 part of cement, having also a proportion, not exceeding 20 per cent. in the case of any block, of large rough stones incorporated in it.

Ballast for Concrete.—The ballast for this work was obtained from the foreshore immediately adjacent to the site of the work, from that of the Duddon Estuary inside Hodbarrow Point, and from the Hodbarrow Mains, bordering on the estuary. The material from the last-named source, though of suitable composition, was somewhat dirty, and in order to remove dirt from such ballast as was not clean enough for use, it was loaded into tip-wagons when excavated, and run to a tip which was situated between the end of Hodbarrow

Point and the east end of the Barrier. Here it was washed by being turned over by the tide, being afterwards again loaded into wagons and conveyed to the block-making yard. In this way about 20,000 cubic yards was treated.

Manufacture of Blocks.—In the block-making yard a cement-store capable of containing 1,000 tons was provided. The mixing of the concrete was done by machine, two Messent mixers being used, each of 1 cubic yard capacity. The ballast and cement were conveyed to the top floor of the mixing-house by a hoist worked by a steam-winch; thence they passed to the mixer on the floor below, which in turn delivered the concrete into small steel tipping-wagons of ½ cubic yard capacity. These were run along staging having an incline of 1 in 100, to the block-moulds, which were on the concrete floor on either side of the staging, of which there were two lines. Moulds were provided for forty blocks, and floor-space for making twice that number, a mould being removed from a block when set and moved on to the site for making the neighbouring block. The moulds were usually loosened after 48 hours and removed after 4 days, and the block was lifted and conveyed to the stacking-ground after about 10 days, the minimum being 7 days. No block was set in the work until it was 4 months old, and stacking-ground was therefore provided for 1,100 blocks. A full week's output from the block-makers was fifty-six blocks, being ten per day for five days of the week and six on Saturdays. This output was naturally not always attained, but block-making having been commenced in January, 1901, by September in the same year, when the first blocks were deposited in the work, there were a large number of well-matured blocks in stock.

Block-setting.—For dealing with the blocks in the block-yard, and for setting them in the work, Goliath cranes of 60 feet span were used. The blocks were lifted for stacking and loading by means of lewis-bars, but for depositing them in the work nippers were used. The cranes for setting the blocks were carried on staging formed of iron girders carried on dolphins of four piles each. The bays of this staging were 40 feet in span and sufficient ironwork was provided for six such bays. The piles for the dolphins were driven before the tipping of the limestone bank, and the inner row of dolphins was included within the limits of that bank; consequently these piles, after the superstructure of the staging had been removed as the work progressed, could not be drawn, but were cut off flush with the bank. Most of the dolphins in the outer row were drawn. The centre-line of the staging was on a curve of 2,780 feet radius, and the staging was erected in successive chords of 240 feet,

special dolphins of six instead of four piles being provided for sluing the Goliath at the end of each length.

When blocks were required for setting they were taken out along the limestone bank alongside the staging to the Goliath, which lifted them, passed them through an aperture in its side frame, and duly set them in place. The day's work at block-setting varied very considerably, being affected by such things as the amount of preparation that had to be done at the foot or on the sides of the limestone bank to receive the blocks, the slow rate of setting which was attainable at points where blocks had to be manipulated under girders, and so on. The best day's work accomplished was fifty-one blocks, but it was seldom that more than twenty to twenty-five blocks were set in a day.

The total number of 25-ton blocks set in the work was 5,157; and in addition to these the two hundred 20-ton blocks which had protected the weakened portion of the first sea-wall were removed from their position and placed in the new work after the closing of the Outer Barrier.

After the removal of the staging and the cutting-off of the piles of the inner dolphins, the spaces left by the tops of these dolphins were filled with blocks tipped from wagons off the limestone bank. Over the top of the block-work, which was at a level 5 feet below the top of the bank, being thus 10 feet above high water of ordinary spring-tides, a covering of rough limestone, similar to that which protects the shoreward ends of the embankment, and of the same width (15 feet), was deposited.

Scour in Front of Tips, and Preventive Measures.—As the two ends of the banks progressed seaward, considerable scour began, as had been anticipated, to take place immediately in front of them, caused by the flow and ebb of the tide around the ends. In order to prevent this, a covering of a mixture of clay and debris from the mines was spread in advance of the work. This mixture was of a very heavy nature and it also consolidated soon, becoming eventually quite hard and affording practically complete protection against scour. An apron of similar material was also spread at the foot of the wave-breaker blocks over the length where the sea-stroke was heaviest, and prevented the abstraction of sand from this place and consequent sinkage of the blocks.

Inner Bank.—The inner bank was intended, as already stated, to be formed of iron slag, and a considerable length was executed in this material. It was found, however, that the weight of the slag was not sufficient to prevent its being clawed down and squandered whenever seas of any strength occurred, and its use was discontinued

in favour of limestone until the sea had been excluded by the closing of the temporary timber dam to be described later, after which a return to the use of slag was made for the remaining portion of this bank. The inner bank was 9 feet wide at the top, with slopes of 1 to 1, and was deposited by end-tipping wagons, the landward side being subsequently trimmed to the desired slope.

Clay Filling.—When these two stone banks had been tipped for sufficient lengths, and the hollow before described had been excavated between them and filled with puddle, the clay bank followed. For getting the clay, two steam-navvies were employed during the greater part of the construction, and a third later on, and a considerable quantity was also excavated by hand, all the clay for puddle being obtained in this way, in addition to a portion of the ordinary filling. The material was deposited in a similar manner to the stone bank, commencing with two end tips, the valley left between the two tips being afterwards filled by side tipping, which was also carried on from the stone banks. This clay was chopped, watered, and rammed as it was deposited. Between 900 and 1,000 cubic yards was usually deposited daily in the work.

Puddle Wall.—Simultaneously with the tipping of the ordinary clay filling the construction of the puddle wall was proceeded with. The wall was brought up in steps of 1 foot, at a slope slightly flatter than the end of the tip, the ordinary clay, side-tipped from off the stone banks, being carefully packed and rammed against its sides as it was raised. The puddle was conveyed by wooden shoots from the top of the bank to the wall, and was deposited therein by hand, being well trodden in to render the mass homogeneous. The wall was 7 feet wide at the foot where it sprang from the puddled trench, or from the puddled hollow around the heads of the piles, and was continued at this width up to 5 feet below high-water level of ordinary spring-tides, when the width was reduced to 5 feet, the wall being carried 10 feet higher at this width.

Surface and Side of Clay Embankment.—The surface of the clay filling, which varies in width between 43 and 58 feet, is finished with a layer of slag 6 inches thick. For this purpose material of small size was selected as far as possible, any pieces which were too large being cracked by hand after spreading, and the whole surface was well blinded. The coating was well rolled with a heavy roller. It was important that it should have a good surface, in order that water should drain towards the seaward side, with which object the inclination of 1 in 40 was given to the surface. The slag coating was not applied until the clay had been deposited for several months; indeed, the landward ends of the

barrier had been finished for more than 12 months before any slag was deposited. Time was thus allowed for settlement of the clay to take place; but notwithstanding this, settlement did take place at some spots after the slag had been laid, and at these places the slag was taken up, additional clay was deposited to make up the hollows, and the coating was relaid.

Soiling and Sowing Slope.—The slope of the clay embankment from its landward edge to the inner stone bank was originally intended to be 1½ to 1, and the major portion of the work was approximately tipped, and a short length trimmed, to this slope. It was found, however, that there was difficulty in getting the clay to stand at this slope, and it was accordingly modified to 2 to 1, material being taken from the top and put to the lower part of the slope, the quantity of clay thus not being increased. It was also decided that this slope should be covered with soil—of which there was available a large quantity stripped from the fields whence the clay was obtained—and sown with grass-seed, to aid in maintaining the surface of the slope.

Diversion of Swash Channel.—It will be seen from Fig. 3, Plate 4, that, although the area to be enclosed, and the site of the Barrier itself, were left almost entirely dry at low water, there was one channel, known as the Swash channel, which still existed after the receding of the tide, and that this channel crossed the line of the work at about the centre, and again near the east end. Between the making of the survey and the beginning of the work some change had occurred in the position of the channel, the eastern point of the crossing having moved much nearer to the shore. It therefore soon became necessary to deal with this. Piles were driven and a staging was thrown across the channel, in the line of the inner stone bank, and from this material was tipped which formed the base of that bank. Clay was also tipped between the two stone banks, and through it the puddle trench was subsequently sunk. In order to divert the channel, and throw it outside the line of the work, material was next conveyed across the channel, and low groynes were formed, consisting of the mixture of mine-debris and clay before referred to. The first groyne was at an angle of about 60 degrees to the work, and extended 100 feet seaward of it, being about 16 feet wide at the top and 3 to 6 feet high. Next, a similar low bank was tipped in the line of the work, just beneath the future foot of the main limestone bank at its seaward side, being thus eventually entirely incorporated in the work. At 250 yards further seaward a second groyne, 350 feet in length, was thrown out at an angle of 35 degrees to the work.

These groynes had the effect of gathering sand and training the channel seaward, and the bank along the edge of the site of the work being continued right round to join the west arm of the embankment, the channel was eventually thrown entirely outside the line of work. Subsequently three similar groynes, each 250 feet long, 400 feet apart, were constructed near the centre of the work, to encourage the deposition of sand there; and they are having the desired effect.

SLUICE-CULVERTS (Figs. 11, Plate 5).

The four sluice-culverts for discharging water through the Barrier from the reclaimed area are situated near the centre of the work, and are formed of concrete, the landward and seaward ends being faced around the openings with granite, as are also the faces entering the penstock-chambers; while the remainder of the face-work of the end and wing-walls is of limestone. The coping around the tops of the penstock-chambers is also of granite.

The centre-line of the penstock-chambers is on the line of the cut-off, and the chambers are 5 feet 6 inches wide transverse to the culverts, and 4 feet 6 inches wide in the line of them. The length of the culverts seaward of the penstock-chambers is 130 feet 6 inches, and landward thereof 59 feet 3 inches. Aprons of concrete, 10 feet wide at the seaward end and 14 feet 6 inches wide at the landward end, were formed; the total length of concrete foundation being 218 feet 9 inches. The width of the seaward portion of the culverts is 4 feet 6 inches, with 3-foot walls between and 3-foot 3-inch outside walls; their height is 5 feet 6 inches. They have a semicircular top and a square bottom, and the inverts have a fall of 2 inches between the penstock-chambers and the mouth. The inner culverts are 4 feet wide and 5 feet high, with semicircular top and bottom, the walls being 3 feet 6 inches thick; the invert here is level, and is 21 feet below high water of ordinary spring-tides. The concrete of the foundations was intended to be of a thickness varying between 6 feet 1 inch and 6 feet 9 inches, which it was assumed from the borings would allow the work to be founded on the clay bed. This thickness, however, was increased in parts, owing to irregularities subsequently discovered in the clay. The concrete of the arches is 3 feet thick. All the concrete for the culverts was mixed in the proportion of 5 parts of shingle and sand to 1 part of cement, and was mixed by hand on the spot. When the subsidiary borings on the line of the cut-off were taken, one of them was situated on the centre-line of the culverts, and clay

was proved at about the anticipated depth. Nevertheless it was deemed advisable, before beginning the execution of the foundations, to make additional borings; accordingly ten bore-holes were put down over the area of the work and apparently showed a practically level bed of clay to exist, no variation exceeding 1 foot being found. The work was therefore begun by enclosing the area with 4-inch sheeting, and dividing it into ten compartments in the same manner, so that the excavation could be undertaken in sections, each 22 feet long, and of the full width of the foundations.

Excavation was commenced in compartment No. 1, at the landward end of the work, and operations had not proceeded far when irregularities in the clay bed were found to exist, the bed apparently entirely disappearing at the north-west corner of the work. On further extension of the excavation it was discovered that between two lines of borings just previously taken there was a complete break in the clay bed, a gully of silt 25 to 30 feet in width crossing the work. As it was not practicable to found the culverts on this, it was decided to pile this portion of the foundations: accordingly, piles were driven 4 feet apart over the area of the gully and extended into the clay bed on either side. The piles in the first compartment were driven to 15 feet below the normal level of the bottom of the foundations, and those subsequently driven were put down 7 to 8 feet deeper. On the heads of these piles, secured by dogs and spikes, were fixed whole-timber cap-sills, which were crossed at intervals by half-timbers, the whole arrangement forming a grid. The heads of the piles were at the normal level of the foundation, which was here carried 1 foot 6 inches deeper. Numerous additional borings were made, and no further great variations in the clay bed were revealed, though when the excavation of compartments Nos. 9 and 10 came to be executed, a fissure 4 feet wide, extending from the seaward face 25 feet into the work, was discovered, lying between two rows of bores 10 feet apart. This was bridged over with a flooring of half-timbers, 12 feet in length, and a protective boxing of sheet-piling was subsequently driven in front of it, to confine the silt.

The conditions described above raised apprehensions as to the efficacy of the cut-off already provided at this spot, which had been made by carrying the 25-foot 6-inch steel piling across the site of the culverts; it was deemed advisable to provide additional protection here, and accordingly an auxiliary line of steel piling, of the 31-foot 6-inch length, extending across the foundations and for 11 feet 9 inches on either side thereof, was driven 21 feet seaward of the

original cut-off, to which it was joined by return lines at the ends. The corners were turned, and the junctions with the existing line of piling were made, by means of gauge-piles which had no angle-bar jaws on one flange, and which were driven with this flange fitting into the breast of the neighbouring gauge-pile, to which they were at right-angles, the flange being chipped to shape for this purpose.

Where the piling crossed the foundations of the culverts, both gauge- and sheet-piles were made with removable upper lengths, which were taken off when the subsequent excavation had been carried out. On this being done the piling was found to have driven accurately and to form a very efficient cut-off. Around the heads of the piles thus left, a recess 3 feet 6 inches wide was formed in the concrete, which was carried to an additional depth of 3 feet, the bottom being 2 feet below the head of the pile, allowing 1 foot for possible settlement. This recess was filled with puddled clay. A similar arrangement had been carried out in the case of the first line of piling. In addition to the extra piling described above, it was necessary to form a boxing of steel piling immediately landward of the original cut-off at the east side of the foundations, in order to enclose a piece of extremely bad ground which existed here under a corner of the work. This was done with 25-foot 6-inch piles.

Where the work was founded on clay the concrete was carried down 6 inches into it. In depositing the concrete over the portion where no clay existed, great trouble was experienced owing to water springing up through the silt, this water apparently finding its way from a pool which remained inside the line of the barrier. In order to deal with this it was found advantageous to lay down a carpeting of felt immediately before the concrete was deposited; this was weighted with stones, and the concrete was laid upon it as quickly as possible. The concrete of the foundations was usually deposited in two layers, the successive sections having a horizontal bond of 5 feet, while keys were formed vertically to prevent lateral sliding. The upper layer was brought up to 6 inches below the invert-level, the top skin in the culverts being put on subsequently. After the culverts had been constructed, the concrete work was surrounded, above shore-level, with a covering of puddled clay, 4 feet thick seaward of the penstock-shafts, and 3 feet thick landward thereof. The puddle wall also divides where it meets the penstock-chambers and is brought around the sides of them, making a casing of puddle, 4 feet thick, on either side. The puddle casing, where it was under the site for the wave-breaker blocks and the outer and inner

stone banks, was covered with a rough paving of stones, as large as could be handled by men, packed to as great a height as was practicable before the banks were tipped across it, so as to prevent stones from piercing through the puddle and damaging the concrete. These banks were brought across the work with great care; the outer bank was lowered at the end as it approached the culverts until it was 8 feet below finished level, and on reaching the side of the culverts the stone was drawn down by hand on to the work, then more stones were carefully tipped, and the same process was repeated until the culverts were crossed. The inner bank was deposited in the same manner, and the first portion of the clay filling was side-tipped from these banks, the puddle casing around the penstock-chambers being brought up at the same time. In this way the weight of the embankment was imposed gradually upon the concrete work. Notwithstanding these precautions, unequal settlement took place to a slight extent, and cracks appeared in the concrete of the culverts where junctions in the concrete of the foundations had been made. None of these cracks was large, and the settlement was very slight. It was considered desirable, however, to put a lining into those portions of the culverts where the cracks were most pronounced, and, at a later period, after the closing of the Barrier, and when all movement appeared to have ceased, a ring of blue brick was built as a lining to these parts.

After the embankment had been raised to its full height and the wave-breaker blocks had been deposited, a parapet 8 feet 6 inches in height, formed by two tiers of concrete blocks, was constructed on the limestone bank over the culverts. It is 96 feet in length on the upper course of blocks and 102 feet on the lower course. The blocks were in the first instance placed temporarily in position and allowed to remain for some months, with a view to consolidating the bed on which they rested. They were subsequently lifted and re-set permanently with cement-mortar joints in the usual manner.

Attached to blocks in the lower course of the parapet, on the landward side, are the winches for raising the tide-flaps at the seaward end of the culverts. The chains from these are led over four guide-pulleys in each case, one on each edge of the upper block of the parapet, one on the edge of a wave-breaker block halfway down the slope, and one on a bracket fixed to the face-wall immediately over the door.

The penstocks fit in cast-iron frames let into recesses in the granite of the inner face of the penstock-chambers, and are raised and lowered by square wrought-iron rods, to the upper ends of which are

attached shrouded racks of cast steel, which are worked by crabs at the top of each chamber. The penstocks are so designed that two men can lift them when there is a head of water of 8 feet above the bottom of the culverts on the inside; while four men would do the work with a head of 20 feet on the sea side. They would of course be used under the latter condition only in the event of its being desired at any time to flood the reclaimed area.

The landward end of each culvert is provided with a wrought-iron grating fixed in a rebate in the granite facework.

At each side of the concrete apron of the seaward end are placed concrete blocks of the same dimensions as the wave-breaker blocks, forming wings. These were built in situ and are held in position by cramps of railway-rails which were built into the concrete of the apron with their ends projecting up into the blocks.

Closing of the Barrier.

It had been foreseen that as the ends of the embankment approached each other the inrush of the tide through the aperture would tend to produce scour immediately inside the Barrier, and that it might become necessary to adopt protective measures here. These anticipations were verified, the scour being very considerable and causing the pool, of which mention has already been made, to be deepened to an extent that was considered to threaten danger to the work; as it was feared that if this action were allowed to continue, the surface here might be lowered so as to be below the level of the foot of the steel sheet-piling. Accordingly, in order to prevent further denudation, an apron of clay covered with limestone and mine-debris was deposited, the surface of this apron being on a level, at its front, with the sill of the temporary dam which was then in course of construction, and being given a slight upward incline towards the shore. The apron extended shoreward 150 feet from the line of the foot of the inner slope of the Barrier, and was carried to some distance on either side of the gap. It effectually protected the ground immediately behind the Barrier; while as a check to the rush of the water, and to prevent, as far as possible, scour from taking place beyond the edge of the apron, mounds of large stones were deposited at short distances apart, in two rows, on its surface. These expedients had considerable effect in the desired direction, for although some scour still took place, it occurred with much diminished rapidity, and in no case reached alarming proportions.

Temporary Closing-Dam (Fig. 12, Plate 5).—The method of closing

the Barrier provided for in the specification for the work was that the ends of the embankment should be advanced towards each other, leaving space for the construction of a dam which should have the necessary sluice-area to admit of the free flow of the tidal water into and from the area to be reclaimed; and the work was carried out in conformity with this plan. Calculations of the requirements were made before the letting of the contract, and further calculations were made after the design of the dam had been decided upon. The accuracy of all these calculations was borne out in the result.

The form of closing-dam adopted was a single-skin timber dam, 292 feet in length, with coffer-dams 65 feet long at each end. The dam was situated slightly to the east of the centre of the work (Fig. 3, Plate 4), and a short distance west of the permanent culverts. It was constructed in the line of the embankment, its face being 5 feet landward of the steel piling, and the banks were advanced so as to house in the dam to the extent of 83 feet at each end, the coffer-dams enclosing the puddle wall.

The quantity of water contained behind the Barrier at high water of ordinary spring-tides was calculated to be 105,000,000 cubic feet, and for its inflow and outlet thirty-six sluices, 6 feet by 4 feet 4 inches, giving a total area of 936 square feet, were provided in the dam, the permanent sluices, which were also taken into account, having an area of 66¼ square feet. The sills of the sluices in the dam were 17 feet below high water of ordinary spring-tides, being 4 feet above those of the permanent sluices. It was calculated that the greatest head of water against the dam that could occur would be 4 feet 9 inches on the flood-tide and 3 feet 6 inches on the ebb, and the greatest head actually observed during the few days when the water was passing through the sluices was 4 feet on the flood-tide and 3 feet 7 inches on the ebb, the height of the tide on this day being 1 foot 6 inches below that of ordinary spring-tides. The top of the dam was 8 feet above high water of ordinary spring-tides The main piles were 16 inches by 16 inches, and were driven 24 feet into the ground or 15 feet into the clay bed. The sheet-piles, which were 6 inches thick, were driven so as to extend not less than 10 feet into the clay.

The sluices were arranged in six bays of six sluices each, with a counterfort between each bay. Where the sluices occurred, and for the portion enclosed in the banks, the main piles were spaced 4 feet 4 inches apart, and at the counterforts 5 feet 4 inches apart.

After the main piles and the sheet-piles had been driven, and during the erection of the superstructure of the dam, the ground between its face and the steel piling was excavated down to the clay

bed and the space was filled with puddle. The hollow between the stone banks was also excavated, and the puddle was deposited, the latter being covered with a coating of mine-debris before fixing the decking of the dam, up to the underside of which it was brought. The decking, which was 5 inches thick, extended for a width of 48 feet in front and 38 feet behind the skin of the dam, and the space between the underside of the decking and the ground was filled with mine-debris over which a layer of clay was laid immediately before the planking was laid down. This was to prevent scour from taking place beneath the decking. The coffer-dams at the ends were formed by driving a row of piles 10 feet 8 inches in front of the main piles and fixing thereon 6-inch timbers placed horizontally: with the 6-inch vertical cleading of the dam, this formed a casing which was filled with puddle in continuation of the puddle wall. The row of piles was continued to form the outer row of the gantry on which the cranes for construction purposes, and subsequently for raising and lowering the sluice-doors, were carried. The inner row of gantry-piles was the same distance behind the main piles of the dam. Two lines of railway were carried on this gantry. The piles of the gantry had triangular cut-water pieces attached to them up to the height of the top of the sluices, to diminish their obstruction to the flow of the water.

The counterforts between the bays of sluice-ways were formed by a system of raking struts at front and back of each pile, and a similar arrangement was made at the back of each alternate pile between the counterforts. Each set of struts was connected together with half-timber bracings, and on the upper struts behind the dam were fixed half-timber horizontal lacings, running the whole length of the work, to prevent racking. As an additional guard against such action, ties formed of steel-wire rope, 1 inch in diameter, were attached to the heads of the counterfort-piles and carried back to those of the piles upon which the feet of the upper raking struts rested, one tie being in the line of the strut and the other being carried diagonally across to the next counterfort. These were tightened by means of union screws.

The cleading of the dam was 6 inches thick, each panel being formed of four timbers over the sluices and five at the counterforts. These panels were put together and caulked on shore and fixed into the work in one piece.

The sluice-doors were 6 feet 8 inches high and 5 feet wide, and were formed of 4½-inch timbers placed horizontally, with two vertical stiffening pieces 9 inches by 4½ inches. A central rib was formed by a piece of rail of 81-lb. section, to the top of which was

attached a rail of 56-lb. section, which, being carried up to the top of the dam, formed the lifting-bar. The weight of these rails acted to prevent the doors from being floated up from their seatings.

At the side of the first counterfort at each end a timber retaining-wall was constructed to keep up the end of the clay bank, which was finished off with a slope of 4 to 1 and covered with a layer of bags filled with puddle to prevent it from being drawn down by the wash of the sea. The ends of the limestone banks were formed of large masses deposited by a crane.

Closing of Sluices.—The construction of the dam was practically completed on the 14th July, 1904, and on the 20th the closing of the sluices took place, the permanent culverts being closed immediately afterwards. Operations were then suspended until the morning of the 22nd, when, all having proved satisfactory, the sluice-doors were caulked and pitched and a beginning was made with stripping the decking of the dam and removing the coating of mine-debris over the puddle in the trench and hollows. At the same time tipping of slag in the inner bank was proceeded with. By the following day arrangements had been completed for continuing the tipping of the main limestone banks, and this was pushed forward, in order to protect the dam, with all possible speed. On the 6th August, when these banks had been brought forward to within 30 feet of one another, a strong gale and heavy sea occurred, but no damage of any kind was sustained. With the exception of the decking, the upper raking struts of the counterforts and the struts which passed through the site of the puddle wall, the whole of the structure of the dam was enclosed in the embankment. The upper portion of the gantry was removed after the clay filling had been brought up. Progress was well maintained, and in October, the Barrier having been raised to its full height all round and being therefore practically finished, the Engineers' certificate of completion was issued. Sundry minor works, such as the levelling of the surface of the more recently executed portion of the embankment and depositing the slag coating thereon, the completion of the trimming, soiling, and sowing of the inner slope, the setting of a few concrete blocks, and other details, were proceeded with during the period for which the contractors were responsible for the maintenance of the Barrier, which period was, under the contract, to cover the six winter months.

Additional Protective Measures adopted after Closing.—After the Barrier had been closed, water was observed issuing in runnels from certain places at the foot of the inner stone bank. It was at first surmised that it might be water which had been held up in the

embankment and land-water draining out through the stone. It was found, however, that while this might account for some of the runnels nearest to the shore, which ran continuously, it did not account for the majority, as it was soon observed that these were affected by the tide, beginning to appear at 2 to 2½ hours before high water, and disappearing at 3 to 3½ hours after the ebb had begun. Moreover, as the tide rose, the surface of the sand inside the barrier for a distance of about 150 feet from it became damp and assumed an appearance of sweating; while as the tide neared its full height, small bubbles appeared, forming small craters in which the sand was kept moving by the water. The force of the water, however, was not sufficient to carry the sand away; nor did there appear at any time to be any material being borne by these bubbles, or by the runnels already mentioned. The sweating usually began about 45 minutes earlier than the runnels, and continued considerably longer. It is a curious fact that these manifestations always began and ended at practically the same length of time before and after high water, although the head of water outside the barrier at such times varied considerably with the state of the tide as regards springs and neaps. At spring-tides, however, the runnels and bubbles were more active and the sweating extended over a wider area.

These indications appeared over a length of about 1,000 feet, behind that portion of the Barrier where the cut-off was formed by 34-foot 6-inch steel piling at the west end of the work, and consequently where the ground was known to be most unreliable; and it was believed that the weight of water outside the Barrier caused the moisture with which the sand was charged to be forced under the steel piling and to the surface inside. In order to check this action it was decided to apply a plaster of clay over the spot, making the weight of this sufficient to counterbalance the pressure on the sea side of the embankment. A patch of clay was accordingly deposited, covering with a good margin that part of the ground where the dampness and sweating had been observed. The depth of water at high water of spring-tides outside the Barrier at this part is 15 feet 6 inches, and at the highest recorded tide it would be 20 feet. The height of the clay filling was therefore made 10 feet 6 inches against the side of the inner stone bank, and a gradient of 1 in 60 was given to the surface, giving an average height of 9 feet. The weight of the clay being 125 lbs. per cubic foot, this gave a weight of 1,312 lbs. per square foot on the ground at the foot of the embankment, or an average weight, over the whole area covered, of 1,125 lbs. per square foot, the pressure of the water outside being 1,280 lbs. at the highest recorded tide and 992 lbs. per square foot at

high water of ordinary spring-tides. The surface of the clay was covered with a coating of slag in a similar manner to the main bank. Since this clay patch was deposited no indication of sweating beyond its boundaries has been observed, and only a very small quantity of water has been seen to issue from the stone bank at a point a short distance below the end of the patch.

In order to prevent the ground immediately in front of the culverts from being scoured away by the necessary periodical openings of the sluices a small apron of mine-debris has been deposited there.

Lighthouse, etc.—The Barrier has been completed by the erection upon it of a lighthouse and fog-signalling apparatus. A single line of railway running the whole length of the Barrier will remain permanently thereon.

Particulars of the quantities, plant employed, etc., are given in the Appendix.

The first sod was turned by Mr. H. Arnold, Chairman of the Hodbarrow Mining Company, Limited, on the 27th April, 1900; the sea was excluded from the area by the closing of the dam on the 20th July, 1904; and practical completion of the Barrier as an efficient sea-defence occurred on the 1st October, 1904.

The Engineers for the work were Messrs. Coode, Son and Matthews, MM. Inst. C.E. The Author acted as Resident Engineer during the construction, having as his principal assistants, Messrs. H. C. Lobnitz, Assoc. M. Inst. C.E., H. G. Mitchell, and A. D. Keigwin, Assoc. M. Inst. C.E.; the two former in succession and the latter during the whole period of construction.

The Contractors, Messrs. John Aird & Co., were represented by Mr. Wm. Liddle, M. Inst. C.E., as chief agent, who had Mr. B. G. Lloyd, Assoc. M. Inst. C.E., as his chief assistant during most of the period, and as engineering assistant, Mr. T. Partington, Assoc. M. Inst. C.E.

The Author desires to express his thanks to Messrs. Coode, Son and Matthews for their kind permission to make use of drawings and other papers in connection with the work; to Mr. Cedric Vaughan, Managing Director of the Hodbarrow Mining Company, who furnished the information relative to the mines which forms the introductory portion of this Paper; and to Messrs. John Aird & Co., whose representatives kindly supplied him with many valuable particulars.

The Paper is accompanied by eight drawings and three tracings, from which Plates 4 and 5 and the Figures in the text have been prepared; also by sixteen photographs. [Appendix.

APPENDIX.

Main Dimensions of the Barrier.

Length	1 mile 530 yards.
Extreme height	40 feet.
Extreme width at base	210 feet.
Extreme width at top	83 feet (including protective coating).
Area reclaimed	170 acres.

Quantities of Principal Materials Used.

Limestone	621,000 cubic yards
Slag	34,000 ,, ,,
Clay	543,000 ,, ,,
Concrete	80,000 ,, ,,
Steel piling	4,416 tons.
Pitch-pine piling, etc.	39,000 cubic feet.
Pitch-pine in dam	60,000 ,, ,,

Explosives Used in Quarrying Stone.

Gelignite	58,200 lbs.
Powder	588,000 ,,
Fuse	39,100 coils.

Principal Plant Employed.

13 locomotives.
3 steam-navvies.
3 25-ton goliath cranes.
2 12-ton locomotive steam-cranes.
4 6-ton ,, ,, ,,
2 4-ton ,, ,, ,,
3 3-ton ,, ,, ,,
4 portable engines.
2 concrete-mixers.
30 piling-engines.
8 steam-pumps.
2 pug-mills.
31 vertical boilers.
25 steam-winches.
12 rock-drilling machines
363 tipping-wagons.
87 ballast-trucks.
24 block-trollies.

Number of men employed during greater portion of period of construction, 1,200.

Discussion.

The President. The PRESIDENT moved a hearty vote of thanks to the Author for his Paper, which described a novel mode of dealing with a very difficult piece of work.

The Author. The AUTHOR exhibited a number of lantern-slides illustrating the progress of the work.

Mr. Matthews. Mr. WILLIAM MATTHEWS, Vice-President, desired first to congratulate the Author on his interesting Paper, and to thank him for the full details given in it of the difficulties which had been encountered, and the procedure adopted to overcome them. He also wished to refer, as no mention was made of them in the Paper, to two incidents which occurred in the early stages of the design of the work. When his firm was instructed to prepare the designs for the work and to report upon it to the Hodbarrow Mining Company's directors, they thought it desirable, in justice to the company and in view of the novel character of the work—particularly the difficulties with regard to the bottom and to making the work water-tight—that a second opinion should be taken with regard to the engineering features of the undertaking, especially with regard to water-tightness. They consequently recommended the directors to obtain such an opinion from the late Mr. James Mansergh, Past-President Inst. C.E. Mr. Mansergh very kindly undertook the duty; he visited the site and went very carefully into the matter, subsequently reporting to the directors that he thought the proposed scheme might be adopted with a fair prospect of success. The directors were then satisfied with regard to the scheme proposed, but the representatives of Lord Lonsdale, who held the mineral rights on the foreshore, thought they also would like to have a separate engineering opinion, and they consulted Sir Benjamin Baker, Past-President Inst. C.E. The plans and reports were laid before Sir Benjamin, who visited the site; and he also reported in favour of the work. Mr. Matthews thought it was only just to the engineers he had mentioned that he should make reference to them, and he desired to thank them for their kind co-operation in carrying out the work. With regard to the form of the works, the mode of procedure adopted by the Hodbarrow Company in carrying on their mining-operations would be described by Mr. Vaughan, the Managing Director; but, shortly stated, the plan adopted was that ore 80

feet to 100 feet thick was taken out and the whole ground was allowed to subside; the working proceeding from the land end seaward. The result had been that in 1884 he was requested, in the absence of Sir John Coode, to report to the directors on the sea-defence works, which at that time were the sole sea-barrier to the property. As the result of the abstraction of the ore, and the consequent subsidence of the ground on which the sea-defence works existed in 1884, those works now lay at a depth of 60 feet below high-water mark. The sea-wall subsequently constructed, which enclosed an area of about 25 acres, consisted of concrete in mass, with a facing of concrete blocks and a backing of clay, the wall being lined immediately at its back by a puddled wall. The wall and embankment were completed in 1890, and until 1898 the work was absolutely water-tight, and no trouble was experienced in connection with the mining. But, one morning, a hole about 60 feet in diameter was discovered on the foreshore, and it was found that a connection had been made between the tide outside and the sand seam which existed in the mine about 120 feet below the foreshore-level. The result was that a considerable quantity of water got into the mine, and there was great danger of the mine being flooded. Fortunately, however, one morning, in a few seconds, the clay backing to the wall dropped, and shut off the water as effectually as would have been done by a sluice. That subsidence of the clay gave his firm the idea of making an embankment of a flexible instead of a rigid character, as had been the case with the old wall. Although the backing dropped about 4 feet 6 inches in a few seconds, the sea-wall was held up in front of it. As the Author said, it showed some signs of distress; nevertheless, it had not altered to the present day since the time when the scour occurred at such a considerable depth underneath it. Before the Outer Barrier was designed, a more than usually numerous set of borings was made by the Hodbarrow Mining Company, who had in their employ a staff of skilled men, accustomed to making borings on their property. As would be seen from the longitudinal section, if the engineers had purposely placed the borings so as to deceive the contractors with regard to the level of the clay bed, he did not think they could have placed them in better positions. It would be noticed that they came at the tops of ridges, and between the borings in some cases very considerable declivities in the clay bed were afterwards found to exist, one of which ran down to a depth of 43 feet below low water of ordinary spring-tides. In carrying out the older work the same kind of bottom, boulder clay, was met with; two or three places were discovered where the clay had pinched out, and a bed

Mr. Matthews.

Mr. Matthews. of running silt had to be contended with. But the difficulties connected with this first work were infinitesimal compared with the difficulties experienced with the bottom in the case of the Outer Barrier. As described by the Author, changes took place at almost every few feet of the work in the character of the clay bed and in the existence of the live-silt strata, which sometimes came to the surface, and was sometimes met with a few feet below it. In designing the Barrier, the first main object in view had been, to provide a heavy structure which would compress the stratum of sand on which the work had to be constructed. For a length of about 1,200 feet, where the clay apron was shown in Fig. 3, Plate 4, there was no water-tight bottom at all, and the work had to be constructed on sand. It was very important to make the structure of such a character at that place that, in the event of any creep occurring underneath the seat of the bank, the bank would settle down and stop the creep so as to prevent a blow underneath; and he believed the design had been successful in doing that. The Paper described what occurred after the completion of the work in the way of "sweating." Mr. Matthews had seen the same kind of thing once or twice before, but in the present instance the dampness covered a large area. The area of the clay apron was about 4 acres, and the effect of the apron had been, he believed, to stop completely the slight sweating due to the variation of the head of water in consequence of the tidal rise and fall. One peculiarity about the work was that it was exposed to a variable pressure. If it had been exposed to a constant pressure, possibly an engineer would have hesitated somewhat before carrying out the work in the way in which it had been done; but inasmuch as the pressure was intermittent, and the extreme pressure existed only at high water, it was thought by his firm when they first proposed the design—and the idea was supported by the late Mr. Mansergh and also by Sir Benjamin Baker—that that mode of construction would be perfectly safe. Again, they designed the cut-off of steel sheeting of such a length that they calculated it would not be possible, looking at the weight on the top, for water to creep underneath, and he believed experience had shown that that idea was correct. Another peculiarity about the work was that it not only had to be water-tight, but also had to resist moderately heavy seas. Thus the difficulties, both of construction and of design, were considerably greater than in making an ordinary water-tight embankment. The Barrier was about $1\frac{1}{3}$ mile in length, and the maximum head on it was about 22 feet at high water of spring-tides, and about 15 feet at high water of neap-tides;

and, as the Author mentioned in the Paper, the quantity of water Mr. Matthews. excluded at high water of spring-tides was about 105 million cubic feet. The sheeting was a most interesting feature of the work. The engineers had decided to adopt that sheeting for the following reasons. The present was the third contract his firm had had the pleasure of carrying out for the Hodbarrow Mining Company in conjunction with Messrs. Aird, and in the previous work, before 1890, some difficulty had been experienced in getting the piling down. Cast-iron sheeting was accordingly tried, but with only moderate success, the sheeting being broken by the boulders in the clay. Wrought-iron sheeting was therefore tried as an alternative, with complete success. In consequence of that, his firm had gone to work with the data obtained from the former sea-wall, and had devised the new steel sheeting for the Outer Barrier. As it was a novel mode of construction, he had thought it important—looking at the interests involved and the extent of the work—that, before proceeding with the steel sheeting, an experiment should be made with a length of it, in ground corresponding with that into which the permanent sheeting was to be driven. It was fortunate that such an experiment was made, because it showed exactly where the defects of the original design were, and after careful examination it had been possible to rectify them. The result was that a perfectly successful steel sheeting was obtained, and a mode of construction which could be recommended with confidence to any engineer who had similar work to do, as affording a very satisfactory cut-off. The closing of the Barrier had been a very anxious job, owing to the enormous quantity of water which had to be excluded. The date selected for the closing had been governed by the considerations that, first, it was most desirable to do it in the summer, when the weather was likely to be fairly fine, and secondly, it was of extreme importance to close the dam at a neap-tide when the water was very low. Both those conditions had been fulfilled: the work had been successfully closed by dropping the thirty-six sluices, as the Author had explained, and on the tide rising again the Barrier had been found to be practically water-tight. The little runnels to which the Author referred were extremely small on so large an area, and for all practical purposes the work was tight before the clay apron was put there. Permanent sluices had to be provided, inasmuch as the rainfall of the district was 45 inches per annum. With that rainfall over 170 acres of foreshore, which also formed a catch-pit into which the land-drainage came, it was necessary to provide means for letting off the water. Mr. Vaughan had informed him that he was now arranging to pump out the leakage-water and rain-water so as to close the sluices permanently. The Hodbarrow Company did not

Mr. Matthews. like the idea of the sluices being there in the event of complications arising with the workmen, or in the event of a strike taking place, because possibly damage might be done to the sluices, through malice, which would involve danger to the ore-workings. These workings, which were about 300 feet below the surface of the ground, had now extended beyond the sea-wall, and the time was not far distant when that wall would share the fate of the former sea-defence works. He thought very interesting data would be obtainable when that wall began to break up and settle, with regard to the strength of concrete beams and of unsupported concrete. He had therefore arranged with Mr. Vaughan to divide the wall into sections, so that accurate observations might be made of what occurred, with a view to introducing into the Proceedings at some future time, as an addendum to the present Paper, the actual results produced in the settlement of the wall when it broke up. One thing not mentioned in the Paper which he thought ought to be stated was that the total cost of the work was about £560,000. Few private companies, he considered, would have the courage to undertake a work of that kind; and now that it had been successfully carried out he trusted the directors would meet with the reward which was due to them. In embarking on such an enterprise, where so many risks had to be taken, he thought it was a great comfort to the engineer to be allied with a good contractor. In the present case his firm had been specially fortunate. Messrs. Aird had command of an excellent staff and of unlimited plant, and all their resources had been placed at the disposal of the engineers. At one time he believed there were as many as thirty-two steam pile-drivers at work, and innumerable pumping-appliances. The contractors' interests had been looked after by Mr. William Liddle, one of the most capable agents it was possible to have on works; and the engineers had been fortunate also in having a good resident engineer, as the members would gather from what they had seen of him that evening.

Mr. Vaughan Mr. CEDRIC VAUGHAN remarked that probably the question which was uppermost in the minds of all the members was, why did the Hodbarrow Mining Company build two embankments when one would have sufficed? He would explain shortly how that had come to pass. The irregularly shaped area on the right of the plan (Fig. 1, Plate 4) indicated the first discovery of ore. When that ore was found, Lord Lonsdale granted the company a lease of the minerals extending far away to the west, beyond the limits of the plan, and also far to the north, the southern boundary of the lease

being high-water mark of ordinary spring-tides, beyond which the company had no power to go. When the ore had been extracted from the larger deposits, the ground subsided at a slope of about 1 to 1; thus it was impossible for the company to work up to the boundary of the lease, otherwise the sea would have been let into the broken ground; and it was necessary to leave a belt of ore 360 feet wide, which at that part of the deposit was about 100 feet thick. Thus a very large quantity of ore had to be left, to protect the workings from the sea. Drift-ways in solid ore were carried to the boundary, and it was found there was a very considerable quantity of ore left on the boundary, clearly proving that a body of ore existed beyond it. Permission was sought from Lord Lonsdale to bore on the foreshore for the purpose of ascertaining what ore was lying there, but he was not disposed at that time to grant permission, for reasons which were best known to his advisers. The company therefore had to be content for a time, and to work on the ore they were able safely to extract. But, at the same time, the sea was gradually eating into the sandbanks on the shore, and a timber revetment had to be driven for the purpose of stopping the erosion. As the Author said in the Paper, one day, during a very severe gale combined with a high tide, the timber revetment was breached, and it then became quite clear, both to Lord Lonsdale and to the company, that something would have to be done to protect the mines from the sea. The company thereupon suggested to Lord Lonsdale that if he would not allow them to explore for the mineral seaward, he should allow them to build an embankment to enable them to work the ore up to the boundary. After considerable negotiations, permission was granted, and Sir John Coode, who had advised the company on other occasions, was called in; the first sea-wall was designed and constructed, and it answered its purpose, in that it enabled the company to work the iron ore right up to the boundary of the original lease. Subsequently Lord Lonsdale granted permission to bore on the foreshore, and accordingly bore-holes were put down in sectional lines on the enclosed area, and it was found that an enormous deposit of ore existed underneath, which was amply sufficient to pay for a much larger work. The directors were considering in their own minds what was the best means of enclosing the area—because it was quite clear that it would have to be enclosed before the ore could be worked—when the unfortunate mishap occurred to which both Mr. Matthews and the Author had referred, namely, the sea got down underneath the foreshore and into the mines. That caused the directors great anxiety at the time, but it eventually proved a Mr. Vaughan.

Mr. Vaughan. blessing in disguise; for it not only convinced Lord Lonsdale and the shareholders of the company that something would have to be done, but was more effectual in hastening negotiations than any arguments the company might have put forward. The result was that within 2 years of the incident the Outer Barrier was designed, the contract let, and a new lease of the mines granted. It might be asked whether the company was sure that, even with the new Outer Barrier, all the ore-bearing ground had been enclosed. The company was not sure on that point, but they had been limited by the Board of Trade as to the area which the Outer Barrier should enclose. Not only the Mining Company but also the Iron Company had a wharf in the estuary, and the navigation of the Duddon was an important one. The ironworks sent pig-iron from their wharf to all parts of the coast, and his own company sent ore to the Clyde and elsewhere. The Board of Trade accordingly would not allow the company to take the Barrier out farther; so that they had been obliged to be content with the work on its present site. Drift-ways into the solid ore had now been made about halfway between the old wall and the new one, as well as cross drifts towards the east. How far the ore extended he could not say yet; but it would be observed from the section that the conditions seaward were altogether different from the conditions on the land side. As the old wall was approached, the workings came under a limestone roof, which became thicker as it proceeded seaward. The section (Fig. 2, Plate 4) was taken through the centre of the plan and went rather to the west and south; if the section had been to the east and south it would have shown a much larger quantity of ore seaward. In working the ore seaward the miners had a rock roof over them, and it was very doubtful whether that roof would cave in; personally he thought it would not. In all probability the ore would have to be worked from the outside first, leaving the voids behind and working inland, instead of, as at present, working seaward; but that remained to be seen. It might interest the members to know how the ore was worked. The shafts were situated outside the ore-ground; a drive was then made underneath the ore, and a rise was made up into the ore, in which there were three main heights of working. By that means they had succeeded in draining the superincumbent water. In the first instance the ore was highly charged with water, and the company tried to sink shafts in the ground, but failed. When he came to the mines in 1872, he sank shafts altogether outside, went down underneath the ore, and rose up, so tapping the water and draining the superincumbent ground. By adopting that plan,

when the pillars were taken out, the ground subsided, and it was found to be dry. The same system was now being followed, but he questioned whether the ground would cave in at all when they came farther seaward and got under the thick rock roof. If the ore extended underneath the Outer Barrier and beyond, it must be left to posterity to find some way of working it. There was sufficient ore in the enclosed area to keep the company going for the next 30 years or more, and with that he thought they would have to be content. He could not speak too highly of the way in which Mr. Matthews had designed the work. He had been asked to build a water-tight embankment, which would also be sea-proof, on ground which was practically nothing but sand. He thought most engineers would have hesitated to undertake such a job, but Mr. Matthews was not dismayed. He made up his mind to solve the problem, and Mr. Vaughan thought the members would agree that he had solved it very well indeed. The Barrier was to all intents and purposes perfectly water-tight; the company was now working in the enclosed area; and Mr. Vaughan hoped some day they would get their money back. Mr. Vaughan.

Mr. MAURICE FITZMAURICE desired to ask one or two questions with reference to the most important part of the work—the cut-off. The puddle-trench at each end, of a maximum depth of about 13 feet, and in which the cut-off was formed by a 5-foot thickness of clay, keyed into the clay at the bottom of the trench, had no doubt proved rather difficult and troublesome work, having to be done under tidal conditions; and it appeared that a long length, which it was originally intended should be done in this way, was abandoned in favour of a cut-off formed of pitch-pine piles or steel piles. Similarly a long length of cut-off, which it was originally intended to form by means of pitch-pine piling, of a maximum depth of about 28 feet, had been abandoned in favour of steel piling. The impression left on his mind, after reading the description of the different kinds of cut-off, was that, if the work were to be done over again, probably both the clay and the pitch-pine piling would be abandoned in favour of steel sheeting. It would be interesting to know if that was the engineers' feeling. As regarded the cost, he imagined the cut-off of clay in a trench, under tidal conditions, was more expensive than either of the other methods, and probably pitch-pine sheeting, although the material itself would be cheaper, would be just as expensive as using steel. It would be interesting to know also the rate of work with the different kinds of cut-off. In a recent Paper by Mr. G. H. Stephens[1] Mr. Fitzmaurice.

[1] "The Barrage across the Nile at Asyût," Minutes of Proceedings Inst. C.E., vol. clviii, p. 26.

Mr. Fitz-maurice. reference was made to a considerable length of cast-iron sheeting forming a similar cut-off. That work also had been carried out by Messrs. John Aird & Co., and he believed the first use of cast-iron sheeting was due to Mr. Matthews. Perhaps Mr. Ellis, who had had experience of both works, could say something on the relative advantages of cast-iron and steel sheeting in different places. Probably where there were boulders steel sheeting was preferable, but in sand cast iron might be more suitable. Mr. Ellis might also be able to give some idea of the relative cost of the two methods of doing the work. It would be interesting to know how deep steel sheeting could be carried—whether it could easily be driven 50 feet or whether 34 feet was found to be about the practicable limit. He would like also to know whether the consolidating effect on the ground under the bank contemplated in the design had been actually experienced to any extent; whether, for instance, any movement which indicated such compression had occurred in the ground at either toe of the bank. It was stated that there had been a certain amount of sweating inside the bank where the piles were deepest and the ground worst. Could any movement or dislocation of the piles have occurred there, owing to settlement of the sand under the weight of the bank? He would have thought that clay would form the best cut-off where settlement might be anticipated, but that remained to be seen. Thanks were due to the Author for the large amount of detail he had given in the Paper. With regard to the cost of the work, taking the quantities of the bank as given in the Table and subtracting such matters as sheet-piling and pitch-pine piling at ordinary rates, the total cost mentioned by Mr. Matthews (£560,000) would work out at between 5*s*. and 6*s*. per cubic yard, which seemed to be a very reasonable price for work of that kind, even when a good deal of the material was found on the site. The only other point upon which he desired a little further information was whether the south-westerly gales, which, it was stated, blew strongly on that shore, had caused any damage to the concrete blocks, which were comparatively light. It would be an advantage to have a further illustration of the timber closing-dam, as Fig. 12, Plate 5, was hardly so detailed as the other figures.

Mr. Ellis. Mr. Basil P. Ellis much regretted the absence of Sir John Aird and Mr. Aird in consequence of a domestic bereavement. The most interesting point about the work was undoubtedly the cut-off of steel piling. As far as he knew, it was one of the largest systems of steel piling that had ever been carried out as a cut-off, and the results showed that it had been entirely successful. In fact, he did not think any other system would have been successful, having regard

to the large boulders found in the foreshore, at varying depths, some nearly as deep as the piling went. With reference to the points raised by Mr. Fitzmaurice, cast-iron sheeting had been used at Asyût, in Egypt, with great success, but there the bottom consisted of silt and therefore the driving had been easy. These piles were mostly 16 feet in length. In the case under discussion, however, cast-iron piles could not have been used, they would have broken when they came in contact with the boulders. At Hodbarrow everything depended upon the spacing of the gauge-piles, and it was wonderful how accurately the gauge-piles and sheeting-piles, 31 feet long, could be driven with very little guidance. All the piles had been driven with only one row of walings, so that the driving had needed great care. The gauge-piles had had to be driven within about ½ inch of their exact positions, for it was clear from the section (Figs. 7, Plate 5) that if they had departed from the perpendicular much more than ½ inch, or 1 inch at most, either way, then either the sheet-piles would not have fitted into the grooves to the right depth, or else there would have been difficulty in getting them down. As a matter of fact, very few of the sheet-piles had jammed. With regard to the question of driving longer lengths of that section, tests had been made more recently at Avonmouth, and he doubted from the results whether they could be driven successfully up to 50 feet or 60 feet. But at all events it had been possible at Millom to drive them to a depth of 34 feet in really awkward ground. He had no particulars of the cost with him, but regarding the rate of progress Mr. William Liddle, who had had charge of the work throughout for the contractors, would furnish some information. Had a clay-puddle trench been possible through the whole length of the Barrier, it would doubtless have been quite as good as the sheeting, but it was not possible, because in some parts there was nothing to key it into. Although the concrete blocks were not novel they had proved highly successful in breaking up the seas, so that very little water, except in the form of spray, passed over the top of the Barrier, even when it was blowing very hard. He had been very glad to hear from Mr. Vaughan that the trouble which occurred at the hole shown on the plan had been entirely stopped by the Outer Barrier. That hole had been carefully filled with clay puddle and stone before the Barrier was made, and since the completion of the Barrier the stream of water which formerly flowed into the mine from that hole had entirely stopped, so that the sheeting was thoroughly efficient. The Barrier had been most interesting work to carry out, under the able guidance of Mr. Matthews and Mr. Bidwell, and he was sure that the members would join with the Hodbarrow Company in feeling satis- Mr. Ellis.

Mr. Ellis. faction at its success from a commercial point of view, as well as interest in it as a unique piece of engineering.

Dr. Deacon. Dr. GEORGE F. DEACON considered that the most valuable part of this valuable Paper was the description of the difficulties met with and the manner in which they had been overcome. The oozing of water in runlets at the inner toe of the embankment, underneath the area where clay was afterwards placed, was a phenomenon which he had observed in more than one place. It was stated in the Paper that no material was removed by those runlets; but he would like to ask whether the Author was quite certain of that. Some years ago Dr. Deacon had been called in to examine a reservoir, in the outer toe of which much the same thing had occurred, except that the runlets were more concentrated, there being three of them, much larger than those at Hodbarrow. It was stated that no material was being removed. He went down to the work and found the usual signs of a leaky dam, depressions at the surface of the top of the dam which had been filled up many times, precisely over the places where the runlets would naturally be expected to pass through the bank and puddle wall. All who were accustomed to go about waterworks embankments knew that that was a not uncommon feature. It did not appear to him possible that those depressions could occur and be filled up so many times without material being removed from below. It was impossible to conceive that the water, merely because it was flowing very slowly through those places, carried no material and yet caused special settlement where the rest of the embankment was intact and showed only the natural settlement which occurred in every embankment after it was constructed. Some experiments were made, and it was found that the smallest possible quantities of silt were being removed. All the puddle was taken out at great expense, and a fact was discovered which had been observed in other cases, but which might be new to many members. The manner in which a leak of this kind increased and became serious was exceedingly interesting. The first evidence of it might be a very slight and unobserved percolation of water from the toe, but this gradually increased in volume and became more concentrated. If such a leak were followed back as far as the puddle trench, and if it were found to cross the puddle, the manner in which it did so was very instructive. If the commencement of the leakage was recent, the cause would be found at once in some local defect, such as the washing in of sand during bad weather, or even more palpable evidence of the neglect of contractors and inspectors alike. But, whatever the age of the defect, the water would be found to be percolating through a

pipe, as it were, crossing the wall of puddle clay and filled with sand; Dr. Deacon. and if the leak were of long standing it would be found that the pipe of sand, instead of being approximately circular in section, was oblong, with its major axis nearly vertical. If such a leak was new, the pipe of sand might be only ¼ inch to 1 inch in diameter. It was formed by the slow but constant washing out of the fine clay particles from between the grit or sand which the clay contained. In such cases the current was never sufficient to carry the grit away, but, as the very fine silt was removed, something had to take its place. What happened was that the roof of the little sand-filled tunnel continually fell in, the result being to produce in course of time a vertical fissure filled with sand, which, in the particular case he was speaking of, was 44 feet 6 inches high. In the same embankment was another such fissure, 31 feet high, and each of these varied between 3 inches and 7 inches in width. In another embankment, which had leaked for many years, he had seen a dike or blind passage of clean sand crossing the puddle trench diagonally, and varying between 18 inches and 2 feet 6 inches in width, and between 6 feet and 9 feet in height. The fissure 44 feet 6 inches in height had extended to within 2 feet of the level to which it had been found necessary to lower the water in the reservoir. The embankment had stood for 20 to 25 years; but how much of that time had been occupied in forming the fissures no one could say. The point he wished to emphasize was that the fissures were full of clean sand from which every particle of silt had been washed, although the effluent had always been perfectly bright and clear. He was convinced that it was not wise to assume that, because it was impossible to detect by any ordinary experiment that clay or other material was being carried away by the water, it was therefore safe to leave a runlet of that kind. It was not safe, especially in puddle made from boulder clay. In that case attention was required from the very beginning. He did not doubt for a moment that the precautions that had been taken with respect to the runlets under the Hodbarrow embankment were sufficient for the present; he only suggested that the work should be watched, and that if any other runlets of the kind were observed outside the limits of the filled-in clay they should not be neglected. Boulder clay contained sand in all sorts of proportions, from almost pure sand, formed by natural washing of the clay and often forming parts of masses geologically known as boulder clay, down to 5 per cent. or 10 per cent. In the case under discussion the engineers had had three of the most awkward things in Nature to deal with—carboniferous limestone, sand and boulder clay. In natural boulder clay dikes of sand were

Dr. Deacon. often the equivalent of the leakage dikes in an artificial puddle wall; the difference being that, owing to the irregular disposition of the sand contained in the clay and the presence of large boulders, they assumed all sorts of irregular shapes, but never, so far as he knew, that of the vertical sand-filled fissures found in well-puddled clay. He noticed that quicksand was tapped in the Hodbarrow mine between the sea and the underground workings in 1898; he suspected that that quicksand was in the boulder clay formation, and he would like to know if he was right. The work was obviously justified by the magnitude of the interests at stake, and he had no doubt whatever that it would be successful so long as the underground workings were not brought too close to the Barrier. Many years must elapse before it was approached, and he only suggested, what he had no doubt would take place, that great watchfulness should be exercised.

Mr. Liddle. Mr. Wm. Liddle observed that the prevailing wind at Millom was from the south-west, and the Barrier had now weathered the storms of two winters and was still intact, not a stone having been moved nor a block shifted. Moreover, during its construction the Barrier had been subjected to one or two very severe gales, one in particular he would like to mention. It occurred on the 27th February, 1903. On the evening of the 26th a south-westerly gale sprang up, and by 5 o'clock on the following morning the wind had attained a velocity of 100 miles per hour. Some members might remember that on that occasion the mail train which left Euston at midnight was overturned with all its passengers on the Leven Viaduct near Ulverston, about 15 miles farther up the bay than the Hodbarrow works. That fact afforded a fair indication of the strength of the gale. On that occasion the Barrier, so far as then completed, came through the ordeal with flying colours: everything remained intact, not even the staging being damaged. Therefore the Barrier must be considered to be sea-proof. He thought that a fair way to look at the cost of it was in terms of the amount of hæmatite ore expected to be won from the reclaimed area, and on that basis the cost came out at about 8*d*. per ton. He did not think that was a high royalty to pay for the benefits received, including the prolongation of the life of the mine for another 35 years. Mr. Matthews had found very good boulder clay, singularly free from sand, which had made excellent puddle and very good filling. He had also found at hand limestone—of somewhat higher specific gravity than usual, owing to the presence of iron—and gravel of very good quality. The fact of those materials being available on the site and free of cost had of course made for economy. In fact, all that had had to be

purchased was cement, steel piles, and a little timber. From beginning to end the gauge of the progress had been the rate of driving the steel piles, and the revised design had lent itself in an eminent degree to rapid execution. After 2 or 3 months' experience, the workmen had got the gauge-piles down very accurately. Indeed, he had no hesitation in saying that if it were possible to reach their points it would be found that the extreme variation between the centres of gauge-piles did not exceed ½ inch. The piling was self-checking. If by any chance two gauge-piles were too far apart at the bottom, the neighbouring piles would be too close, with the result that the sheet-pile could not be driven, as it would begin to bite in the grooves. One or two cases of that sort had occurred, which had been rectified at once. What appealed to him was the very broad point given to the sheet-piles, 1¾ inches wide. He thought that was quite the correct thing, because such points either penetrated the ground or stopped absolutely. If they stopped, the workmen knew immediately that a boulder had been met, and they drew the pile and dealt with the boulder. On no occasion had the points of those piles evinced the slightest tendency to buckle. Had they buckled, it would have been impossible to draw them in the ordinary way. This could only have been done by stripping off the angle-bar and taking out the pile. When it was borne in mind that all the piling had had to be done as tidal work, and that on many occasions, at low water of neap tides, it had been impossible to work longer than about ½ hour, it could be understood that it was highly essential to have a simple system of piling; and he ventured to say that Mr. Matthews's design could not be improved upon. On many occasions the full battery of forty boilers, six or seven locomotives, and 400 or 500 men went down to drive piles and found the tide had not receded from the piling-line—in fact, that the tide was beginning to flow again—and they all had to come back, having incurred heavy expense without doing a stroke of work. He was sure that but for the simple design of sheet-piling the Hodbarrow Barrier would not have been finished in the contract time, instead of being completed well within it, notwithstanding the facts that a good deal more work had been done than was originally contemplated, and that there had been delay incidental to changing the design from the puddle trench to steel piles. Mr. Vaughan had intended to say a few words about the system adopted at Hodbarrow for winning the ore, and at his request Mr. Liddle would give some details of the method. The ore-body at Hodbarrow was of very varying thickness, but the average might be taken as 80 feet, with a maximum of about 100 feet. It was worked in Mr. Liddle.

Mr. Liddle. three horizontal planes, about 25 feet vertically from floor to floor. The work was pushed ahead on the upper plane first of all; parallel headings were driven, 38 feet between centres and 8 feet wide, and at every 38 feet cross drifts were driven, thus blocking out the area after the fashion of a chess-board. Rises were then driven; they were 8 feet by 5 feet, and were carried up to the underside of the overlying limestone; then drifts were driven through those again, so as to form the tops of the pillars. The pillars left by this system of headings and cross drifts were 30 feet square, and those at the different levels were vertically over each other. The next process was to rob the pillars. The ore was extracted in slices of 9 feet at a time—9 feet wide, 9 feet long, and 9 feet high; and as the ore was taken out, larch bars were put in to support the roof temporarily. By-and-by these bars yielded, thus allowing the roof to subside gradually. The process was carried on in that manner from the roof of the ore body right down to the floor, and the result was the broken ground shown in Fig. 1, Plate 4. The whole of the workings were worked from the land side towards the sea. When the Outer Barrier was within measurable distance of being finished—about a year before it was actually closed—Lord Lonsdale gave permission for an extension of the old headings under the old sea-wall towards the new barrier. By the time that barrier was actually closed those headings had extended out about 700 feet, all in solid ore. Now the headings had extended out for 900 feet, and on the west side of the barrier the extreme limit of the ore-body had already been reached; but on the east side, at a distance of 700 feet from the central heading, the limit had not yet been reached, and the heading was still in good solid ore. He thought that must be very satisfactory to the owners of the mine. The pumping-plant was excellent. The three pumps, of the Cornish type, had cylinders 70 inches in diameter and 9 feet stroke, and were capable of dealing with 2,000 gallons per minute. Until about 10 years ago the amount of water usually dealt with was about 4,000 gallons per minute, but at the present time it was only about 1,200 gallons. Mr. Vaughan had informed him that during the last 12 months the amount of water to be dealt with had considerably decreased, so that he thought Mr. Matthews must be congratulated on having built a water-tight barrier.

Mr. Hawksley. Mr. Charles Hawksley, Past-President, had listened to the Paper with great pleasure, because it appeared to him to be a model of what such a Paper should be. It described not only the object of the work, but also the successive steps which had been taken to achieve

that object, and in particular it gave a clear description of all the difficulties encountered, and how they had been overcome by degrees. He gathered from remarks made by Mr. Vaughan that it was intended to work the ore by the long-wall system, as it was called in coal-mining. That, of course, must have the effect of letting down the bed of the sea, and, even if the sand should descend, as it probably would, it might be that the mines would become flooded. Mr. Vaughan considered it was sufficient to look 30 years ahead; of course, that was a long time in the life of a man, but perhaps not so very long in the life of an important undertaking such as that of the Hodbarrow Mining Company. Therefore, it had occurred to him that, in view of the possibility of the company obtaining a lease of additional ore on the seaward side, it might be desirable that, before letting down the bed of the sea, at least two galleries should be formed by leaving in ore, or in some other way supporting the bed, so as to provide access hereafter to any additional take of ore that might be secured to the south of the present limit. That might not be very difficult of achievement, although it would necessarily be somewhat costly; but, having in view the amount of ore that might be obtained, it was perhaps worthy of consideration. Mr. Hawksley.

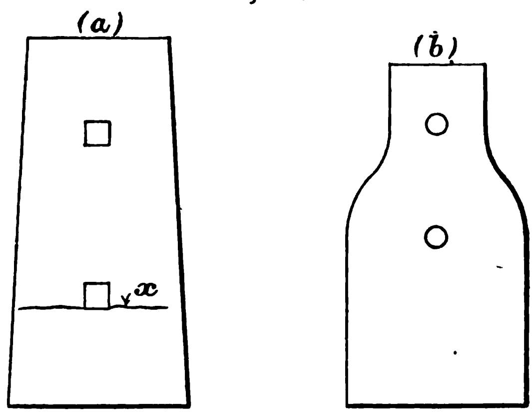
Mr. JAMES C. INGLIS remarked that the old sea-wall pointed to the fact that even engineering firms of long standing became wiser as they grew older; for it was clear that the design of that wall was open to improvement, and that it would have been quite impracticable to carry it farther. He desired to know whether, before making the large sacrifice in connection with the clay apron, consideration had been given to the question of the possibility of attempting to stop the runlets by steel piling or otherwise. He was of course aware of the difficulties in dealing with the sea on the outer side of the Barrier; but it would be interesting to know if the possibility of some alternative to sacrificing 4 acres inside the area, at a place where the ore was fairly deep, had been considered. Another point he had noticed was the somewhat ingenious arrangement of the big blocks in front of the sea-wall, which was intermediate between the old method of putting blocks one on top of the other with geometrical precision and the pell-mell system adopted by the French in breakwaters. He thought that, by hitting the mean, Mr. Matthews had made an important improvement in using blocks economically, because had they been thrown pell-mell many more would have been required, and no doubt they would have cost more to maintain, as such blocks were never very permanent. He had noticed several times in the harbours of the Mediterranean that blocks displaced by heavy Mr. Inglis.

Mr. Inglis. storms were constantly having to be replaced. The arrangement adopted at Hodbarrow had overcome much of that trouble. He thought the cost of setting the blocks in that fashion was well worth the advantage which it gave. The interesting remarks of Mr. Ellis and Mr. Liddle had suggested to him the question whether experiments had been made in driving the steel piles with monkeys of greater weight than usual. The piles were quite different from ordinary piles, and perhaps, having regard to their indestructible character, heavier strokes than usual might have been applied. The precautions taken to ascertain beforehand the character of the ground seemed to have been very elaborate, but the longitudinal section showed that they might with advantage have been much more so. No doubt many engineers were acquainted with big disputes, in which many unkind things had been said, arising out of the freaks of ground between successive borings. Although the Paper had not increased the reputation of borings, it would cause engineers to look more kindly upon them when they did prove a little misleading.

Mr Brown. Mr. P. B. Brown remarked that if there was any fault in the excellent Paper under discussion it was the fact that some of the difficulties in connection with the pile-driving had been somewhat glossed over. It was stated that difficulty had been experienced in driving the steel piles owing to the bolts and fastenings of the monkeys soon becoming crystalline and breaking, and that this difficulty had been overcome partly by introducing rubber cushions between the fastenings and the monkeys. It had been his fortune to be responsible for the manufacture of some of the monkeys used in the work. At first monkeys of the ordinary steel type were used, but they lasted only a short time. Then Messrs. Hadfield were asked to manufacture similar monkeys in cast steel, and those also broke very soon. It was suggested that possibly they were not sound, and great care was therefore taken in the execution of the next order; but these also broke. A number of monkeys were then made, using three different kinds of steel; one an exceedingly mild steel, with about 0·2 per cent. carbon; an intermediate quality, which corresponded with the first, with about 0·35 per cent. carbon; and another with about 0·7 per cent. carbon. Strange to say, all three kinds lasted only for a very short time. It could well be understood, however, that the jar on the monkeys was very severe when in some cases it took one hundred and sixty blows to drive a pile through 1 inch. He visited the work in progress to look into the matter, and he could not find any defects in the castings; they were perfectly sound in every respect. He could only come to the conclusion that the design was not suitable for such heavy work. The first monkeys were of the usual design,

weighing 30 cwt. each, about 14 inches square at the bottom, and 12 inches square at the top. There were two square holes through each for carrying the tee-bolts acting as guides. In each case the monkey broke across the line marked x in *Fig. 13* (a), and presented a smooth fracture, as if cut by a knife, no crystals showing at all: this indicated that the fracture was due to severe compression. After thinking the matter over, he came to the conclusion that the jar on the monkey was so severe as to destroy the adhesion of the crystals of the steel, and, in order to overcome the difficulty, he made a suggestion which fortunately worked out well. He suggested that the difficulty might be got over by adopting a principle well known to manufacturers of projectiles, namely, keeping the weight as far as possible in the head. Mr. Brown.

Figs. 13.



Some monkeys were made in that way, keeping the two bolt-holes as near the top as possible, and the bulk of the weight below (*Fig. 13 b*). The success of that plan might be judged by the fact that no more monkeys were ordered on the contract; and he was pleased to hear from Mr. Liddle that no further trouble had been experienced. They were made of the same class of steel as those of the first design that failed.

Mr. C. H. Colson pointed out no mention had been made in the Paper of the possibility of corrosion of the piles. Buried as they were under a large mass of material, if corrosion took place at any future time the effects might be very serious. As the filling pile was comparatively thin it did not seem clear that the life of the piling would be very long. In America many kinds of steel piles had been used of late years, especially two called the "Friestedt" and "U.S." steel Mr. Colson

Mr. Colson. piles, which seemed to possess advantages over the pile used at Hodbarrow, because they did not require any guide-piles, one pile following on another without any difficulty. Some of the difficulties found in getting the guide-piles down at such exact intervals might have been overcome by the use of the American piles.

Mr. Inglis. Mr. JAMES C. INGLIS thought, from an experiment he had made, that the peculiar fracture referred to by Mr. Brown might be due to the meeting of vibratory waves. He had experimented on that point by letting a 97-lb. rail fall from a height of 14 feet on to a huge block of cast iron made for the purpose, having on its surface undulations spaced at about the ordinary distance apart of chairs on a railway-track, great care being taken to ensure the rail falling as horizontally as possible. The end of the rail, which was quite a sound one, fractured at the first blow about 18 inches from the end, and the fracture was exactly as had been described—a perfectly clean, fine-grained fracture of Bessemer steel. The only explanation he could think of was that the vibrations set up by the impingement of the rail on the block of cast iron ran to the end of the rail and came back and met others, and that where they met the rail fractured. There was no flaw or impurity to initiate the crack at that particular place. The overhang of the end of the rail was about 16 inches, and the rail broke just over the last projection on the block.

Sir Alexander Kennedy. Sir ALEXANDER KENNEDY, Vice-President, asked Mr. Brown whether he had intentionally shown the holes in the monkey round in one of his sketches (*Figs. 13*) and square in the other.

Mr. Brown. Mr. BROWN replied that the shape of the holes had been altered to prevent the injurious crystallization which occurred when a square core was put in a casting. The holes had been made round, with a square at one end in order to prevent the keys from turning.

Dr. Deacon. Dr. DEACON asked the Author to state whether the particular boulder clay through which the piles had been driven contained large boulders or not.

The Author. The AUTHOR, in reply, could hardly give a definite answer to Mr. Fitzmaurice's question whether in future, should another barrier have to be carried out, puddle trench and pitch-pine piling would be abandoned in favour of steel piling. So far as experience went, the puddle trench and the pitch-pine piling had been perfectly successful, and the portion of the Barrier executed with those materials was water-tight. Nor could he very well say anything with regard to the cost of the piling, as it had depended on so many things. For the same reason it was difficult to compare the costs of the different kinds of cut-off, conditions having varied so greatly.

The rate of progress with the steel piling had been about a pile and a half per tide per engine; in the pitch-pine piling and puddle trench it had varied very considerably. He thought experience showed that it was doubtful whether piles more than 40 feet long could be driven accurately. The Barrier had gradually subsided into the sand as anticipated; a certain amount of subsidence had been allowed for, and there had been no subsidence beyond that. Some very heavy gales had occurred during the progress of the work, and nothing had been shifted. The blocks had naturally subsided a little into the sand from their own weight after being set, but no further disturbance of material had taken place. With regard to Dr. Deacon's remarks as to the removal of material, the runlets had been watched very carefully, and at no time had it been possible to detect anything being carried away. No depression had occurred at any time in the work. He thought it might safely be said that the water was not coming down through the clay. Had it been so, these manifestations would presumably have been found at other points, and not only where the bottom was known to be treacherous. The clay puddle was excellent, and the ordinary clay itself became puddle after a very short time. At various times during the progress of the work pits had been cut into the clay to ascertain how the work had consolidated, and of what character it was; and it had been found that the whole clay hearting had become practically puddle. The boulders in the boulder clay were all of granite and had apparently been ice-borne at some period; there were many indications of ice striation on the limestone laid bare at one part of the work, and the boulders were similar to those which occurred in the Eskdale Hills, some miles away. They varied in size from a couple of fists to 2 or 3 cubic feet. The extremely slow progress mentioned had occurred when a boulder had been met with. When the presence of an obstruction of that kind had been ascertained means had been taken to remove it. The question of dealing with the small runnels had been discussed, and it had been concluded that the plan adopted would be the correct one. No sacrifice of ground was entailed by it, as there was practically no ore under that part of the enclosed area (see Figs. 1 and 3, Plate 4). The monkey first used had weighed 34 cwt., which had afterwards been reduced to 30 cwt. The force of the blow had been settled by trial. The monkeys had been of cast steel at first, and subsequently of the toughened steel described in the Paper and mentioned by Mr. Brown. The Author thought that in driving successive piles without any gauge-piles there would be very little check on the accuracy of the work with nothing on one side. That

The Author.

The Author. had been found in the first experimental bay. The flat sheet-pile, guided only on one side, went out of line, and did not form a successful cut-off. The gauge-piles seemed to have rendered the driving accurate, and to have guided the intervening sheet-piles very closely. He had strong doubts whether piles guided on one side would have been successful. It was not anticipated that corrosion would take place to any serious extent, it having been generally found, he believed, that it did not occur in the case of iron buried in this way. Having had occasion to demolish structures of iron in the sea, he had found that while the upper portion of piles, especially the part within the tidal range, had corroded seriously, little or no deterioration had occurred in the part below the surface of the ground.

Correspondence.

Mr. Brodie. Mr. JOHN S. BRODIE considered the Paper most interesting, as describing what was not only a structure for sea-defence, but also a work which presented to a large extent the problems involved in the construction of a water-tight dam for a reservoir on very pervious strata. It appeared that the borings taken originally at intervals of 600 feet were much too far apart. It was well known to those who had to carry out structures in the proximity of hæmatite deposits that the stratification was almost always very irregular, and borings at such a wide interval would be of little use when the work came to be executed. The experience gained in the actual construction of the Barrier fully bore out what might have been anticipated in this respect. It was open to criticism whether the cut-off under the centre of the clay embankment was really necessary. It must be remembered that the problem in this case was not simply to make a dam, as such, perfectly water-tight; for although a cut-off had been provided by means of a puddle trench tied into the clay, and by the timber and steel sheet-piling, yet, bearing in mind that mining-operations would be carried out under the surface of the foreshore to a depth of probably 10 to 15 fathoms, the water-tightness, either of the dam itself or of its key into the clay, by no means guaranteed the shutting-out of the sea from the mine-workings. Strata might exist underneath the clay bed—as in 1898 in connection with the original sea-wall—which would not be affected by the water-tightness of the dam or of the piling underneath it. It therefore appeared that an

equally successful, and much less costly, plan would have been to rely on a properly constructed clay dam, protected on the outer side by the limestone rubble facing and by the irregular blockwork, to preserve the clay dam proper from the erosive action of the sea, and to rely upon the weight of this dam to cause the closure of any fissures which might develop far below the surface of the foreshore. By so doing, the two primary objects kept in view, namely, the flexibility of the Barrier to enable it to follow, without its own destruction, the gradual subsidence of the surface of the foreshore, and a compressive weight to consolidate the underlying strata of sand, would have been equally well secured at a much lower cost. It did not appear from the Paper for what practical purpose the four sluice-culverts had been introduced at all. When the sea-water had been drained from the 170 acres of reclaimed land, a much less sectional area of sluice than was provided by the four culverts would appear to be ample, and anyone with experience of tidal flaps or other mechanical expedients would much prefer to do without them altogether on an exposed foreshore such as this; or, at all events, to reduce the number to a minimum. Mr. Brodie.

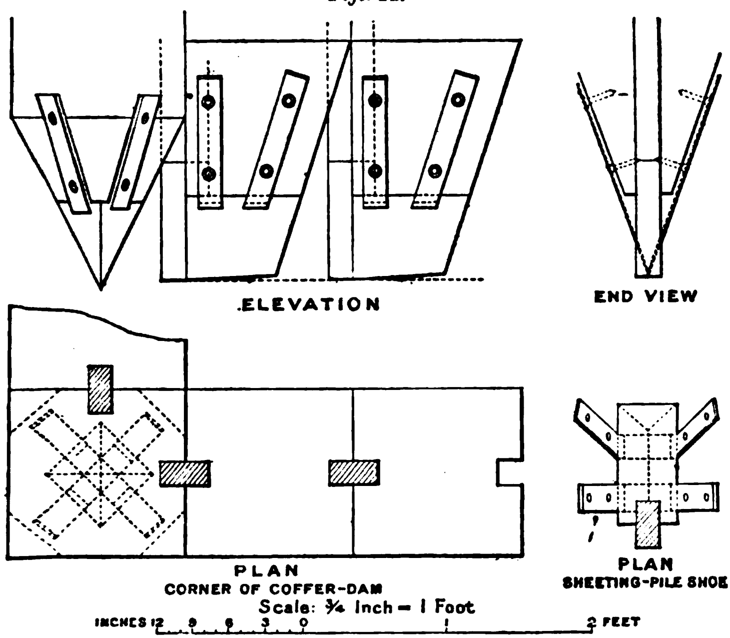
Mr. W. Dyce Cay observed that the necessity for the Barrier to possess some power of adjustment to meet subsidence accounted for the large body of clay in the embankment, which was otherwise undesirable, as it might be damaged by atmospheric action; and this mass had probably led to the puddle wall being made unusually thin. Having had some experience of forming a cut-off in difficult ground, he would have preferred a double row of sheet-piling with a clay puddle wall between the rows. He had used this system in restoring the dock-entrance at Arbroath, which had sunk into quicksand; the borings showing 23 feet of soft sand below the sill-level, then 10 feet of coarse sand and gravel, and then sandstone rock. The following was a description of the procedure adopted. He surrounded the site with a single-skin pine dam, like a large caisson, 79 feet by 64 feet inside horizontally, the sheet-piles before being driven being made up into panels of three, each 2 feet 9 inches wide by 11 inches thick, and 41 feet long. These drove well, except in some cases where obstructions such as boulders or ruins of old works were met with. He tried the water-jet, but in this case it did not prove to be of much advantage. The caisson was, however, well shored inside, additional sheeting was driven behind the defective piles, and he was enabled, after pumping it dry, to get out the main part of the ruins. Then a "blow in" occurred, and he had to use diving-apparatus to remove the remainder. The ruins having been removed, the interior of the caisson was dredged to 17½ feet below the sill, and the Mr. Cay.

Mr. Cay. pit was filled up with clay puddle into which bearing-piles were driven 4 feet apart between centres; and two rows of sheet-piling, 8 feet between centres of the rows, were driven across the work, forming a cut-off under the sill. Then, the piles having been sawn off at the level of the top of the clay, and the whole covered with a sheet of jute cloth, concreting under water was proceeded with up to a certain level, the concrete being carried 5 feet deeper so as to form a tongue between the sheet-piling rows of the cut-off. When this had set, the work was proceeded with by working half tide to half tide, powerful pumps belonging to a salvage-company being used to empty the caisson on the fall of the tide. Some leaks occurred, which were built into tubes and ultimately plugged. On visiting Hodbarrow he had been much interested in the arrangement of the pell-mell concrete wave-breaker blocks, and he hoped the Author would give an authoritative description or sketch of it, which would doubtless be interesting to foreign members. It appeared to him that the lowest course, instead of having the base of its blocks horizontal, had each of the blocks canted in one direction, say from right to left, until its sides were inclined at an angle of 30° to the perpendicular in the longitudinal direction of the Barrier; then those of the course above it were canted in the opposite direction, making a similar but opposite angle with the vertical; and the next course reverted to the same direction of cant as the first, and so on: the slope of the work facing the sea being always maintained at 1 horizontal to 1 vertical. He supposed the reason for the adoption of this class of blockwork was that the waves might be absorbed as well as broken, so that no spray might fall on the clay embankment behind the bulwark.

Mr. Clark. Mr. Alexander Clark remarked that the description given of the experimental piling recalled methods adopted on other works. In the coffer-dams used in connection with the foundations of Putney bridge, which had been designed by him, it had been noted at the beginning of the work that, if a single coffer-dam was to be successful, great attention must be paid to the proper formation of the groove and to the design of the shoe of the sheet-pile. *Figs. 14* showed the formation of the corners of the dams and the adjacent sheet-piles. The 2-inch square plug on the front of the shoe was needed to clear out the open groove left in the pile previously driven. The material was packed very firmly, and on no account must this part of the shoe be brought to a point, as very small stones getting into the groove would lead to the bursting asunder of the timber. The tongue throughout must also be made as tight a fit as possible, no play being allowed. The bevel or "lead" at the point of the pile had also been carefully studied

so as to cause each pile to draw tightly alongside the pile previously driven. The piles in these coffer-dams were 45 to 50 feet long and about 14 inches square, the tongues being 4 inches by 2 inches, and the groove in the piles 2 inches square. They were driven in bays of 12 to 15 feet, a sheet-pile being driven on each side of the guide-pile, all leading towards those already driven until the key-pile was reached. This was made of the exact dimensions to suit the space, was sharpened only crosswise to the dam, and had no lead to either side. The system had been so Mr. Clark.

Figs. 14.



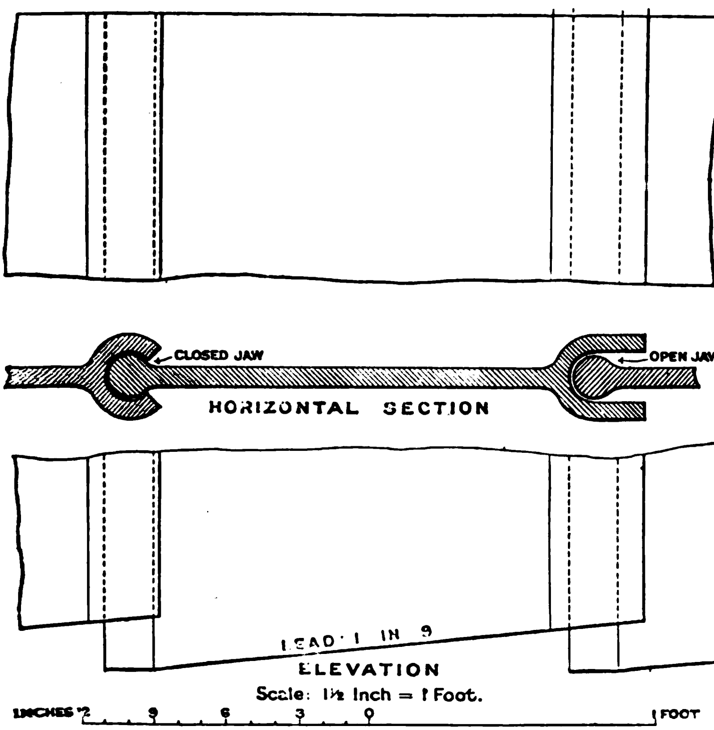
PUTNEY BRIDGE: PILING OF COFFER-DAM.

successful at Putney that the same design had been followed in the foundations of the Battersea bridge, and it had also been adopted for the foundations of the new Kew bridge, by permission of the engineer. In all these cases it had been found well suited to the work, the foundations being in sand, gravel, and clay. There was no drawing in the Paper showing the amount of lead given to the sheet-piling at Hodbarrow, but in Figs. 11, Plate 5, the sheet-piling (which was also tongued and grooved) was shown with diamond-pointed piles. These points did not seem suitable for drawing the piles together so as to make tight joints. In difficult

Mr. Clark. ground it was well not to be encumbered with gauge-piles driven into their permanent position, because there was no flexibility to meet emergencies, and, if there was any part of contractors' work which required provision for contingencies, it was certainly pile-driving. Driving permanent gauge-piles at first necessitated having special sizes for the key-pile on every division, and these were very difficult to get with iron or steel piling. When, however, a gauge-pile was driven temporarily, a uniform size of sheet-pile could be used without any key-pile, as the gauge-pile could be driven permanently into its place after the sheet-piling had been brought up to it. In using iron or steel in that way it was difficult to ensure filling the space accurately; whereas with timber it was not difficult to form a pile tapered or parallel to fill any opening. It would seem to be better to have as few key-piles as possible when steel was the material used. By using a temporary gauge-pile in the manner indicated above, the sheet-piles could be used continuously with far less danger of getting out of line, and with more facility for making a correction, should the pile twist on meeting an obstacle in driving. A good form of steel or cast-iron sheet-pile was shown in *Figs. 15.* It consisted of a solid plate of the required length, about $\frac{3}{4}$ inch thick, having the leading side swelled out to the form of a bead about $1\frac{3}{4}$ inch or 2 inches in diameter, and the other side formed into a jaw or clip to embrace the bead of the pile behind. The jaw at the back of the pile might either have parallel sides or be made so as almost to encircle the bead of the pile behind, and the piles should be made to fit tightly together when driving, without any allowance for play beyond that necessitated by the roughness of the castings, just as the tongues of timber sheet-piles were made to fill perfectly the spaces in the grooves. The closed jaw was preferable if the nature of the ground would allow of its being used, but it might be well to have a few piles with open jaws on the ground in case any correction in plumbing were required. It was mentioned in the Paper that the engines used for driving this piling were of the ordinary type, and subsequently the difficulties experienced with the attachments of the gear were referred to. The ordinary type of pile-driving machinery was of such ancient date that the difficulties connected with its use were almost forgotten. A piling-ram, such as the Lacour or other steam-cylinder design, gave no trouble with monkey-eyes, gauge-bolts and cotters, as the tongue of the ram passed through the leaders and had a very light and simple fastening behind it, which was scarcely ever known to give way. The Lacour machine could strike twenty to thirty strokes per minute at any distance between 1 inch and 6 to 8 feet, according to the length of the ram. When the ram was once set on

the top of the pile, it followed it downwards without any further trouble. The original difficulties with this machine were the breaking of the three-way cock and the bursting of the flexible tube; but these difficulties were very slight in view of the amount of work done, and they had long since been overcome. It was not evident from the description why the sheet-piling should have been cut into half-lengths. It was usually preferred to have the piles as long as possible if the length was not altogether inconvenient, as Mr. Clark.

Figs. 15.



they could thus be more easily controlled and kept in place in driving. There was also a great saving in time by not having to stop work in order to fix on the upper portion of the pile, and considerably more rigidity resulted in the whole structure. The necessity for the use of a dolly was not obvious. Possibly the construction of the piling machines had not admitted of the use of hanging leaders for driving piles below the level of the staging on which the piling-machinery was placed, but no details were given of this staging, and therefore it

Mr. Clark. was difficult to conjecture why there should have been a dolly at all. As a general rule, fully one-half of the effect of the stroke was lost by the intervention of a dolly, and in very hard driving the proportion would be still greater.

Mr. Robinson. Mr. JOHN ROBINSON had been favourably impressed, on visiting the Hodbarrow works, by the admirable arrangements for manufacturing, stacking, and depositing on the sea-face of the Barrier, the large concrete blocks to act as wave-breakers; but it was really the steel sheet-piling in which he was interested, as he had had experience himself with cast-iron sheet-piling in connection with new dock-works. He thought that perhaps steel sheet-piling had some advantage in driving over the cast-iron kind he had used, which had bulb-shaped tongues passing down a groove in the adjoining pile, allowing of considerable play. He did not feel sure that the steel had much advantage, and his choice of steel or cast-iron would depend on the nature of the resistances to be encountered. But to get any kind of pile, whether steel, cast-iron, or pitch-pine, to penetrate hard boulder clay, gravel with water-worn stones, or slag, was very difficult and troublesome. In closing a sea-embankment it was advisable to choose for the purpose a place some distance away from the deepest portion, and, if possible, where the ground would not be injuriously affected by scour. The forming of an artificial floor, capable of resisting erosion, on the site proposed for closing, as had been done at Hodbarrow, and closing the gap afterwards with heavy material, was to be commended. On inspecting the Barrier since its completion, he had observed that it was water-tight, with green-sward on the inner slope; and, although there had been stormy weather recently, he could not see that either the limestone or the concrete blocks on the sea-slope had sustained any damage. It would have been quite different had the clay hearting or the backing with clay been omitted, as the tide, on rising after the closing of the Barrier, would have flowed through the rubble or slag, carrying portions of it away inwards, and a similar thing would have occurred on the ebbing of the tide, doing damage to a less extent. He noticed that in addition to the culverts a steam-pump had been provided for lifting water over the Barrier.

Mr. Siccama. Mr. H. T. H. SICCAMA remarked that it was a well-known fact that the driving of piles through layers of compact sand or gravel was difficult and unsatisfactory, as the resistance and the friction increased as the piles were multiplied. As foundations for heavy structures they were not trustworthy in such soils, as often nothing on the surface indicated whether they were properly driven, or shivered, or deflected. If they were driven with the assistance of

the water-jet the sand was churned up, and in settling it left a space beneath the cap-sills or timber flooring which might lead to disaster. The steel sheet-piling also, in this case, seemed to have been warped and disjointed on passing through the sand. It had lately been reported that in the harbour-works at Rotterdam, where timber piles had to be driven through a layer of coarse sand, tipped in only shortly before to cover fascine-beds, some of the piles, which were drawn owing to their not having been driven true, were found to be not only split and shivered, but were charred, and caught fire on being exposed to the air. It would be interesting to know whether the Author had noticed similar results. If not, it might be that pitch-pine was less liable to such accidents than the Baltic timber used for the piles at Rotterdam. Mr. Siccama.

Mr. JOHN STRAIN remarked that the works were unique, and well designed for the object in view. The feature of the design was its elasticity, to provide against subsidence, or "draw," due to mineral workings. The engineers had evidently had to feel their way in dealing with this difficult problem; and the Author had frankly recounted the failures made before reaching success. The lesson to be taken from this work was the value of steel sheet-piling as against timber, for either temporary or permanent coffer-dam work. Had steel been used from the beginning, instead of timber and puddle, it would probably have saved much money—and certainly much anxiety. It would add much to the value of the Paper if the cost of the work described were stated against the "Quantities of principal materials used" on p. 189. It would also have added to the interest of the Paper had the Author given some rough estimate of the quantity of ore which the works would render available. Mr. Strain.

The AUTHOR, in reply, remarked that the cut-off under the centre of the embankment was necessary to give security against percolation through the pervious strata immediately underlying the Barrier. After the closing of the Barrier, a spring or runnel of sea-water, which had existed in the mine in the line of the inrush of 1898, gradually became less in volume, and it had since ceased to flow. It would be noticed that there would be no mining-operations in the immediate vicinity of the worst ground, the Barrier having been purposely kept away from the scene of operations here (Fig. 1, Plate 4). The sluices provided for the draining-off of water accumulated behind the Barrier were no more than sufficient, having in view the very heavy local rainfall (45 inches per annum), and the facts that one day's abnormally heavy rain would cause a serious accumulation, and that it was possible to keep the sluices open only for a The Author.

The Author. comparatively short period, as they had to be closed well in advance of the rising tide. The arrangement of the wave-breaker blocks was substantially as described by Mr. Dyce Cay, save that the slope was $1\frac{1}{2}$ to 1, and had been adopted for the reasons surmised by him. It had been found quite satisfactory in attaining the objects in view, no heavy water finding its way to the upper surface of the embankment in any storm that had yet been experienced. The timber sheet-piling forming the western portion of the cut-off was provided with shoes somewhat similar to those described by Mr. Clark, only the gauge- and key-piles being diamond-pointed. The timber sheet-piling shown in Fig. 11, Plate 5, referred to by Mr. Clark, was only a small boxing to confine a narrow gully of silt. In the steel piling the system adopted after trial, namely alternate gauge- and sheet-, or gauge- and key-piles, if that term was preferred, had been found to give very satisfactory results. How accurately the piles as a whole had been driven might be inferred from the fact that in only one case in the whole length of $\frac{7}{8}$ mile had it been necessary to use a special closing-pile. The pumping-arrangements for lifting water over the Barrier, referred to by Mr. Robinson, had been provided by the Mining Company, the directors fearing that, in the event of trade disputes with the miners, the culverts might possibly be tampered with. In the case of the Barrier the piles—with the exception of those under the culverts, as to whose satisfactory driving there was no doubt—had not been driven as foundations to the structure but to form a water-tight cut-off. The pitch-pine piles had driven satisfactorily through the sand; it was in the bed of hard gravel, as mentioned on p. 165, that difficulties had been experienced. Though the Author had found many piles split and burred he had not come across any instance of charring. The cost of the Barrier had been mentioned by Mr. Matthews in the discussion, and the quantity of ore available from the newly-enclosed area might be gathered from Mr. Vaughan's statement that the company would be kept going for 30 years or more, and from the average annual output mentioned in the Paper (p. 157).

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27 March, 1906.

Sir ALEXANDER RICHARDSON BINNIE, President, in the Chair.

The discussion upon Mr. Shelford Bidwell's Paper, "The Outer Barrier, Hodbarrow Iron Mines," occupied the evening.

Sect. II.—OTHER SELECTED PAPERS.

(*Paper No. 3592.*)

"A Problem Relating to Railway-bridge Piers of Masonry or Brickwork."

By Frederick Karl Esling.

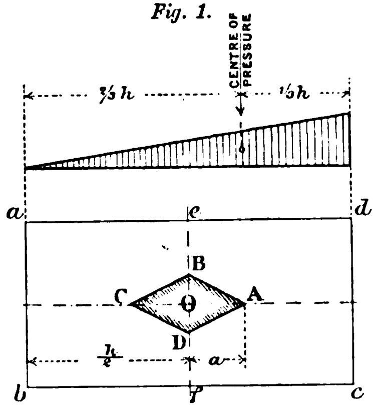
In the design of railway-bridge piers of masonry or brickwork, a puzzling problem is met with when the horizontal forces (due to application of the brake, till sliding takes place, to a train exposed to wind-pressure), acting at an angle to each other, cause the centre of pressure of the combined vertical and horizontal forces to fall beyond a certain distance from the centre of gravity of the horizontal section of the pier. This section is usually square or rectangular in shape, and the following investigation will be confined to piers of these two sections.

The difficulty in calculating the exact stresses resulting in certain cases from the combined action of the horizontal and vertical forces, is due to the fact that masonry and brickwork, even when built in cement-mortar, cannot be relied upon to resist tension. Brickwork in cement is capable of withstanding considerable tension, and cases have occurred of partial damage to ordinary brick-in-cement culverts, in which the portions remaining intact must have been subjected to a tension of 80 lbs. per square inch. But both masonry and brickwork usually contain some joints at which the parts can be separated easily, and in most railway-piers the joint between the pier and its support is quite unable to resist tension.

In *Fig. 1* the horizontal forces are assumed to be acting with such intensity that the compression along $a\ b$ is reduced to *nil*; it will be seen at once that the compression along $c\ d$ must be double the compression due to vertical loads only, and that the centre of pressure will be at the point A. Supposing the line of no-stress to occupy successively every position possible around the rectangle $a\ b\ c\ d$, whilst always remaining in touch with it, then the corresponding centres

of pressure describe the diamond-shaped figure A B C D. This rhombus is called the "kern" (kernel) by German scientists, and the distance of the point A from the centre O is—

Fig. 1.



$$a = \frac{I}{A.\frac{h}{2}},$$

A being the area of the rectangle and I its moment of inertia with $e\,f$ as axis.

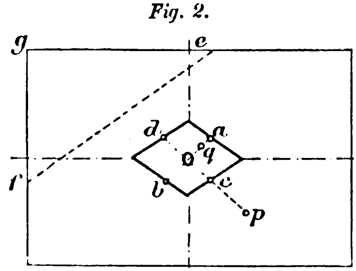
This gives for the rectangle:—

$$a = \frac{b.\,h^3}{12} \div \frac{b.\,h^2}{2} = \frac{h}{6}.$$

The kern therefore occupies the middle third of the two main axes, and with its assistance many of the problems relating to the combined action of the horizontal and vertical forces may be solved.

If the centre of pressure be within the limit of the kern, as at q in *Fig. 2*, it follows that there must be compression only, although of varying intensity, over the whole section of the pier. Let a straight line be drawn through q and O, intersecting the kern in a and b. Then the maximum compression is given by $\frac{W}{A} \times \frac{q\,b}{O\,b}$, and the minimum compression by $\frac{W}{A} \times \frac{q\,a}{O\,a}$, W being the total vertical load on the pier, and A the area of the latter.

Fig. 2.

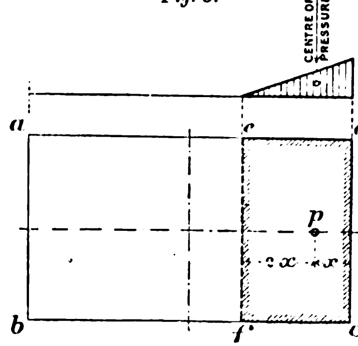


If the material of the pier be capable of resisting tension, the stresses can be found just as easily if the centre of pressure falls outside the kern, say at p, *Fig. 2*. Let the straight line through p and O intersect the kern in c and d. Then the maximum compression will be $\frac{W}{A} \times \frac{p\,d}{O\,d}$, and the maximum tension $\frac{W}{A} \times \frac{p\,c}{O\,c}$. But if the pier be unable to resist tension, a line of cleavage will be formed, say at $e\,f$, so that the area $e\,f\,g$ will be lost, and the compression will be increased on

the remaining part of the original rectangle. Unfortunately there is no direct method of finding this line of cleavage (or no-stress); and the cases investigated in the text-books seem to be confined to those in which the centre of pressure falls on one of the two main axes of the rectangle.

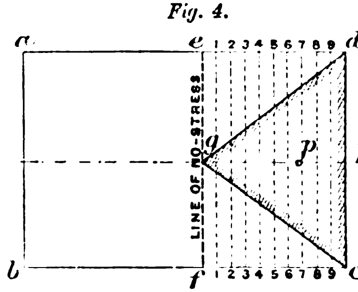
Supposing *a b c d*, *Fig. 3*, to be a horizontal section of a rectangular pier, with the centre of pressure, *p*, outside the kern; then it is known that *e f*, the line of no-stress, is twice as far from *p* as *p* is from *c d*, and that the maximum compression is along *c d*, and equal to $\frac{2\text{ W}}{\text{Area } c\,d\,e\,f}$, or that it is greater than the compression due to vertical loads only, in the ratio of $\frac{2\,b\,c}{f\,c}$. It will be shown that these data are sufficient for solution of the problem under consideration, namely, the determination of the maximum compression on the pier when the centre of pressure falls outside the kern, and away from the two main axes.

Fig. 3.



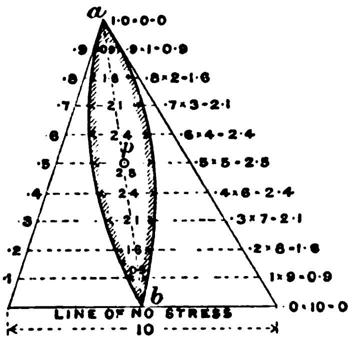
Examining the area *e f c d*, *Fig. 4*, at *e f* there is no pressure, whilst at *c d* there is double the average compression, and the centre of pressure, *p*, is $\frac{1}{3}\,g\,h$ from *c d*. Dividing *d e* into ten equal parts, as shown, and assuming the compression along *c d* to be 10, then along 9 9 it will be 9, along 8 8 it will be 8 . . . along 1 1 it will be 1, and along *e f nil*. Instead of taking the whole area *c d e f* as being active, an "equivalent area" of the maximum compression can now be constructed by marking these reduced lengths 9, 8, ... 1, 0, on the corresponding lines 9 9, 8 8 . . . 1 1 and *e f*. The area *c d g* is thus obtained, and this is a triangle, with the centre of pressure, *p*, on its centre of gravity. In *Fig. 5* is shown the "equivalent area" of a triangular section, with the base as the line of no-stress.

Fig. 4.

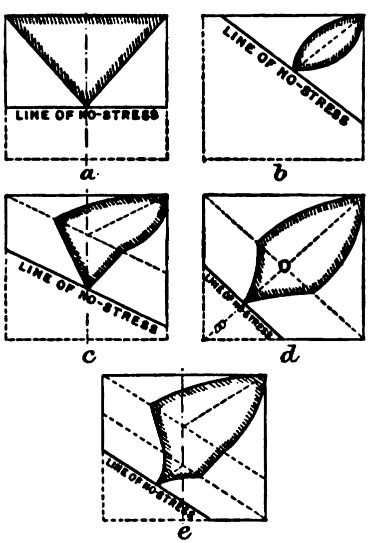


The equivalent area is bounded by two parabolas, is one-third the area of the triangle, and has its centre of gravity on the middle of *a b*. The "equivalent area" provides a simple method of finding the maximum compression, as it is only necessary to divide the total vertical loads by this area. In the case illustrated in *Fig. 4*, the maximum compression is $\frac{W}{\text{Area } c\, d\, g}$. The same reasoning applies to any position of the line of no-stress. Dividing the distance between this line and the farthest part of the rectangle into any number of equal parts by lines drawn parallel to the line of no-stress, and marking off on these parallel lines the reduced lengths, 9 9, 8 8, . . . 1 1, as in *Figs. 4* and *5*, the "equivalent area" can be readily drawn and the maximum compression obtained; the centre of pressure will be on the centre of gravity of the "equivalent area." All the different "equivalent areas" come under one of the types shown in *Figs. 6*.

Fig. 5.



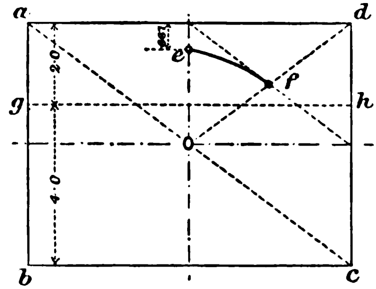
Figs. 6.



If a great number of these "equivalent areas" were drawn and their areas and centres of gravity fixed, lines of equal pressures could be plotted, which would serve to indicate at once the maximum compression caused by the action of the horizontal and vertical forces combined; but any attempt to solve the problem in this manner would soon prove to be an arduous task. What is wanted is a simple method of determining a few lines of pressure, and of deriving others from them.

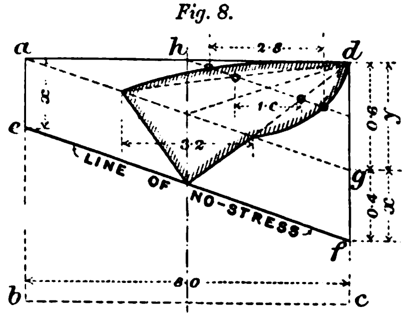
The Author was first led to investigate this problem by finding that the "equivalent area" for a triangle having its base on the diagonal of the rectangle was one-sixth the area of the latter, and its centre of gravity, coinciding with the centre of pressure, was the middle point of the line joining the centre of the rectangle with the apex of the triangle under consideration. It seemed likely that there would be a curve between the centre of pressure of this triangle and that of the rectangle *a g h d* shown in *Fig.* 7 (also giving an "equivalent area" equal to one-sixth the area of the pier), on which would be located the centres of pressure of all intermediate "equivalent areas" equal to one-sixth the area of the pier. For convenience, the terms "6-line," "3·85-line," &c., will be used to denote such curves for equivalent areas of $\frac{1}{6}$, $\frac{1}{\cdot 85}$, &c., of the area of the original rectangle.

Fig. 7.



The term "6-line" will therefore denote the locus of the centre of pressure for all cases in which the resulting maximum compression due to the action of horizontal and vertical forces is six times that due to the vertical forces alone. The calculations were made for a rectangle 8 feet by 6 feet, but any other dimensions could have been chosen. For the 6-line the equivalent area must be $\frac{8 \times 6}{6} = 8$ square feet. Thus in *Fig.* 7, for the rectangle, *g h* is 2 feet from *a d*, with the centre of pressure, at *e*, 0·667 foot from *a d*. For the triangle *a c d* the 6-line centre of pressure is at *f*, the middle point of O *d*.

Fig. 8.



It also seemed very likely that all other areas represented by the 6-line would have one end of their fourth side on *a g* and the other end on *h c*. This assumption proved to be correct, after the following calculations had been made.

Taking a line of no-stress at *e f* (*Fig.* 8), so that *a e* and *f g* are

both $0 \cdot 4$ foot and $d g$ $0 \cdot 6$ foot, the equivalent area is readily calculated from the dimensions given on *Fig. 8*. It consists of a triangle below $a g$, and a triangle plus two areas bounded by parabolas over $a g$. The former is found to be $3 \cdot 2 \times \frac{0 \cdot 4}{2} = 0 \cdot 64$ in area, and the latter $3 \cdot 2 \times \frac{0 \cdot 6}{2} + \frac{2}{3}(2 \cdot 8 - 1 \cdot 6) \times 0 \cdot 6 = 0 \cdot 96 + 0 \cdot 48$. Adding these, the total equivalent area is found to be $2 \cdot 08$ for $d f = 1 \cdot 0$. The co-ordinates of the centre of gravity of this area are $1 \cdot 077$ along $h d$, and $0 \cdot 2603$ at right-angles to $h d$. The calculation shows that the abscissa is a simple function of the length of the pier (in this case always $8 \cdot 0$), and the ordinate a simple function of $(x + y)$. The centre of pressure of any other area with the ratio of x to y equal to $\frac{0 \cdot 4}{0 \cdot 6}$ can therefore be easily determined. For the 6-line (area $= 8 \cdot 0$) x and y are multiplied by $\frac{8}{2 \cdot 08}$, giving $x = 1 \cdot 538$, and $y = 2 \cdot 308$, whilst the centre of pressure will have an abscissa of $1 \cdot 077$ on $h d$, and an ordinate of $0 \cdot 2603 \times \frac{8}{2 \cdot 08} = 1 \cdot 001$ under $h d$.

Evidently similar calculations can be performed for any position of $e f$ (so long as f is on the line $c d$), and a few trials will show that when x is to y as $0 \cdot 4$ is to $0 \cdot 6$, if the centre of gravity of the "equivalent area" is on any "z-line," it will have an abscissa of $1 \cdot 077$ on h d, and an ordinate of $\frac{6}{z} \times 1 \cdot 001$ below $a d$. Thus for the 4-line the ordinate is $\frac{6}{4} \times 1 \cdot 001 = 1 \cdot 501$. For the 12-line it is $\frac{6}{12} \times 1 \cdot 001 = 0 \cdot 50$, &c. An important case is that in which f falls on c, as it is clear that the proportionality to the 6-line will cease when f is anywhere on $b c$ except at the point c. The line $e f$ for this case will be termed the "limit-line." For $d f = 1$ the equivalent area has been found to be $2 \cdot 08$ and for $d f = d c = 6 \cdot 0$, it is clear that the "equivalent area" will be $6 \times 2 \cdot 08 = 12 \cdot 48$, and the limit-line of pressure $= \frac{8 \times 6}{12 \cdot 48} = 3 \cdot 85$. With the ratio of x to y that of $0 \cdot 4$ to $0 \cdot 6$, all lines of pressure between $3 \cdot 85$ and infinity can now be drawn by making the abscissa on h d equal to $1 \cdot 077$, and the ordinate equal to $\frac{6}{z} \times 1 \cdot 001$, z representing any pressure-line between $3 \cdot 85$ and infinity. Similar calculations were made for

various ratios of x to y, and the results obtained are given in the following Table:—

TABLE I.—ABSCISSAE AND ORDINATES OF THE 6-LINE.

$\frac{x}{y} =$	∞	$\frac{0\cdot9}{0\cdot1}$	$\frac{0\cdot8}{0\cdot2}$	$\frac{0\cdot7}{0\cdot3}$	$\frac{0\cdot6}{0\cdot4}$	$\frac{0\cdot5}{0\cdot5}$	$\frac{0\cdot4}{0\cdot6}$	$\frac{0\cdot3}{0\cdot7}$	$\frac{0\cdot2}{0\cdot8}$	$\frac{0\cdot1}{0\cdot9}$	0
$x =$	2·00	1·993	1·967	1·918	1·837	1·714	1·538	1·295	0·968	0·541	0
$y =$	0	0·221	0·492	0·822	1·225	1·714	2·308	3·021	3·871	4·865	6
Abscissa on *hd* . .	0	0·140	0·295	0·466	0·653	0·857	1·077	1·309	1·548	1·784	2·0
Ordinate under *ad* .	0·667	0·702	0·744	0·792	0·850	0·918	1·001	1·100	1·217	1·352	1·5
Limit-line .	2·00	2·21	2·46	2·74	3·06	3·43	3·85	4·32	4·84	5·41	6·0

The foregoing Table gives eleven points of the 6-line of pressure, and by setting out these points the curve can be traced as shown at *ef* in *Figs. 9* and *11*.

Calculating the ordinates below *ad* for the "limit-lines" of Table I, another curve, namely, *if*, *Figs. 9* and *11*, is obtained. The problem is now nearly determined. It will be seen (*Fig. 9*) that four sharply-defined areas, O *im*, *imf*, *ihnf*, and *nfd*, can be marked off, and that these four areas include all positions of

Fig. 9.

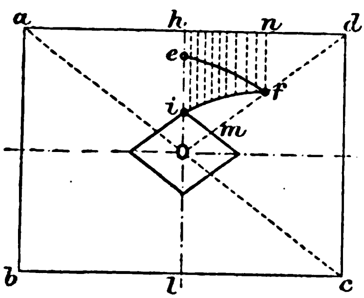
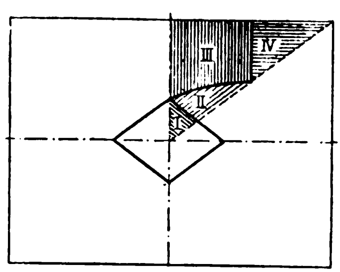


Fig. 10.

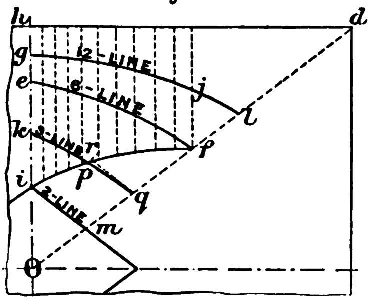


the centre of pressure. In *Fig. 10* these areas are shown shaded. Area I contains the centres of pressure for all pressure-lines between the 1-line and the 2-line, inclusive, *i.e.*, those due to the line of no-stress keeping outside, or just touching, the original rectangle; area II includes all centres of pressure due to the original rectangle being reduced by the line of no-stress to five-sided figures of types *d* and *e*, *Figs. 6*, the pressure-lines ranging from the 2-line to the 6-line; area III contains the centres of pressure for types *a* and *c*, *Figs. 6* (the

2-line to the infinity-line); whilst area IV includes the centres of pressure for type b, *Figs. 6*, ranging from the 6-line to the infinity-line.

It is now necessary to find some additional centres of pressure on the diagonal, O d, *Fig. 11*. For those between f and d this operation is simple, since here only triangles have to be dealt with, and these have "equivalent areas" equal to one-third the area of the triangle (with the line of no-stress on the base of the triangle), whilst the centre of gravity of each "equivalent area" is in the centre of the line drawn from the middle of the base-line to the apex opposite. This means that the centre of pressure for any z-line, where z is greater than 6, will be at a distance from d equal to $f d \sqrt{\frac{6}{z}}$. Supposing it be desired to draw the 12-line complete, the ordinates between g and j are obtained by halving those of the 6-line, and since $d\,l = 2 \cdot 5 \sqrt{\frac{6}{12}} = 1 \cdot 768$, the point l is fixed.

Fig. 11.



To connect j with l a number of triangles are constructed having an "equivalent area" of $\frac{8 \times 6}{12} = 4 \cdot 0$, or an actual area of $12 \cdot 0$, and those for which the centres of pressure fall between l and j are selected. In this manner as many points as may be desired can be obtained, thus completing the 12-line from g to l.

To find the centres of pressure between O and f, a number of lines of no-stress between O and x (*Figs. 6, d*) are assumed, and the "equivalent areas" and distances of their centres of gravity along Of from O are calculated. The results for a number of cases are shown in Table II.

TABLE II.—CENTRES OF PRESSURE ON O f, FIGS. 6 (d) AND 11.

Pressure-lines.	2·0	2·22	2·477	2·773	3·0	3·33	3·645	4·0	4·364	5·0	5·533	6·0
Distance from O along Of	0·833	1·012	1·20	1·389	1·516	1·678	1·815	1·952	2·167	2·263	2·389	2·5

The pressure-lines referred to in Table II can be readily plotted. Thus the points on h O are obtained by dividing twice $h\,i$ by each

pressure and laying off the result obtained along $h\,i$ from h. (This gives for the 6-line, $h\,e = \frac{2 \times 2}{6} = 0 \cdot 667$; for the 3-line, $h\,k = \frac{2 \times 2}{3} = 1 \cdot 333$, &c.) To draw the 3-line complete, the ordinates between k and p are obtained by doubling those of the 6-line; the point q is obtained from Table II; and between p and q a flat curve is plotted by drawing $q\,r$ parallel to $i\,m$, and $p\,r$ parallel to $O\,f$, the curve $p\,q$ to be a parabola with ordinates parallel to $p\,r$. A few trials with equivalent areas of type *e*, *Figs. 6*, will show this curve to be practically correct.

It is now possible to determine any pressure-line between the 1-line and the infinity-line. In *Fig. 11* the 2-, 3-, 6-, and 12-lines are shown; but for the purposes of Table III many more were drawn. The quarter of the original rectangle is divided into smaller rectangles (*Fig. 12*), each one-fortieth the dimensions of the section of the pier. The values of the centres of pressure at the intersections of these lines are determined by scaling between the two nearest pressure-lines; the results obtained are recorded in Table III.

Fig. 12.

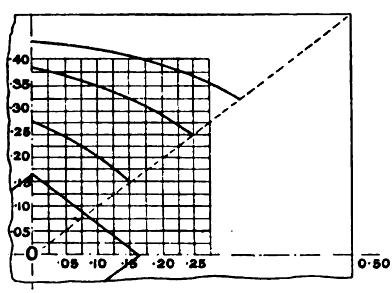


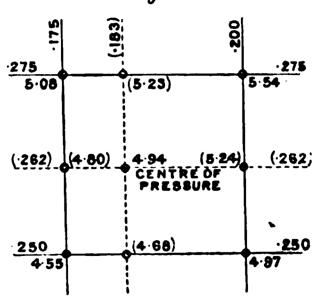
Table III can be used for the solution of most problems involving the action of horizontal and vertical forces on rectangular piers. It is only necessary to find the component of wind-pressure and vertical loads in decimals of the length of the pier, and the component of the brake-force and vertical loads in decimals of the breadth of the pier, and to take from Table III the value of the resultant centre of pressure. As an example, let it be desired to ascertain the maximum compression on a pier 100 feet in height and 25 feet by 12 feet in cross-section, the wind-pressure being 50 tons, the brake-force 50 tons, and the total vertical loads 1,850 tons. The component in decimals of the length of the pier will be $\frac{50 \times 100}{1{,}850 \times 25} = 0 \cdot 108$; and the component in decimals of the breadth of the pier will be $\frac{50 \times 100}{1{,}850 \times 12} = 0 \cdot 225$. In Table III the pressure-line at the intersection of $0 \cdot 225$ with $0 \cdot 100$ is found to be $3 \cdot 27$ and that at

TABLE III.—LINES OF PRESSURE AT INTERSECTIONS OF COMPONENTS OF FORCES.

Component = Breadth of Pier multiplied by	Component = Length of Pier multiplied by																
	0	0·025	0·050	0·075	0·100	0·125	0·150	0·175	0·200	0·225	0·250	0·275	0·300	0·325	0·350	0·375	0·400
0	1·0	1·15	1·30	1·45	1·60	1·75	1·90	2·05	2·22	2·43	2·67	2·96	3·33	3·87	4·44	5·33	6·67
0·025	1·15	1·30	1·45	1·60	1·75	1·90	2·05	2·21	2·39	2·61	2·87	3·18	3·58	4·08	4·77	5·73	7·18
0·50	1·30	1·45	1·60	1·75	1·90	2·05	2·21	2·38	2·58	2·81	3·09	3·43	3·87	4·41	5·16	6·17	7·73
0·075	1·45	1·60	1·75	1·90	2·05	2·20	2·37	2·56	2·78	3·03	3·33	3·70	4·16	4·74	5·56	6·66	8·30
0·100	1·60	1·75	1·90	2·05	2·20	2·37	2·56	2·76	2·99	3·27	3·60	3·99	4·48	5·14	5·99	7·16	9·00
0·125	1·75	1·90	2·05	2·20	2·37	2·56	2·75	2·97	3·24	3·54	3·90	4·32	4·81	5·57	6·47	7·78	9·74
0·150	1·90	2·05	2·21	2·37	2·56	2·75	2·96	3·22	3·51	3·84	4·22	4·66	5·28	6·03	7·04	8·45	10·6
0·175	2·05	2·21	2·38	2·56	2·76	2·97	3·22	3·50	3·81	4·16	4·55	5·08	5·73	6·55	7·66	9·17	11·5
0·200	2·22	2·39	2·58	2·78	2·99	3·24	3·51	3·81	4·13	4·50	4·97	5·54	6·24	7·12	8·33	9·98	..
0·225	2·43	2·61	2·81	3·03	3·27	3·54	3·84	4·16	4·50	4·93	5·48	6·05	6·83	7·82	9·13	10·9	..
0·250	2·67	2·87	3·09	3·33	3·60	3·90	4·22	4·55	4·97	5·48	6·00	6·67	7·50	8·57	10·0	12·0	..
0·275	2·96	3·18	3·43	3·70	3·99	4·32	4·66	5·08	5·54	6·05	6·67	7·41	8·37	9·55	11·1	..	..
0·300	3·33	3·58	3·87	4·16	4·48	4·81	5·28	5·73	6·24	6·83	7·50	8·37	9·37	10·8	..	..	..
0·325	3·87	4·08	4·41	4·74	5·14	5·57	6·03	6·55	7·12	7·82	8·57	9·55	10·8	..	..	..	..
0·350	4·44	4·77	5·16	5·56	5·99	6·47	7·04	7·66	8·33	9·13	10·0	11·1	..	..	..	..	..
0·375	5·33	5·73	6·17	6·66	7·16	7·78	8·45	9·17	9·98	10·9	12·0	..	..	..	..	..	..
0·400	6·67	7·18	7·73	8·30	9·00	9·74	10·6	11·5	..	..	..	..	..	..	..	..	..

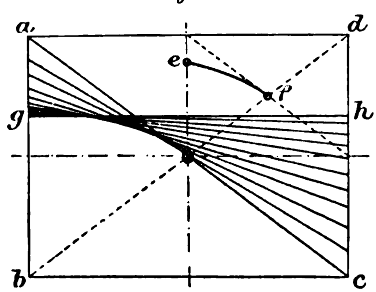
its intersection with 0·125 is 3·54. Dividing proportionately for 0·108, the resulting pressure-line is found to be 3·36, or, in other words, the maximum compression on the pier will be $\frac{1{,}850}{25 \times 12} \times 3 \cdot 36 = 20 \cdot 72$ tons per square foot. Supposing, in another case, the components of the pressure to be 0·262 and 0·183 of the length and breadth of the pier respectively, the line of pressure is found, on reference to Table III, to be in the square shown in *Fig. 13*. Dividing proportionately, vertically and horizontally, the pressure-line is found to be, in both cases, 4·94. If the two results should differ slightly, the mean value is taken.

Fig. 13.



The only method of dealing with this problem previously known to the Author is one which was described by Mr. J. H. Fraser in a Paper on "The Design of Bridges," read before the Engineering Students' Society of Melbourne University in 1892. In this method the centre of pressure is determined in the usual manner, and a trial line of cleavage (or no-stress) is assumed. Stiff paper is used for the drawing, and after the "equivalent area" is marked, it is cut out and balanced on a pin passed through the centre of pressure. Probably the piece cut out will overbalance to one side, and a new line of cleavage is assumed which will counterbalance the area shown to be in excess. The problem is solved when the "equivalent area" cut out balances perfectly on the centre of pressure determined in the first instance. Care and patience are necessary, as several trials are usually required to obtain a satisfactory result. By this method, however, great accuracy is attainable. The example already quoted, having reference to a pier 25 feet by 12 feet, was worked out by this method, and the maximum compression was found to be 20·7 tons per square foot, or the same as that obtained by means of Table III.

Fig. 14.



Referring again to *Fig. 7* and Table I, the following problem presents itself for solution by mathematics or graphical statics. If all lines of no-stress (for the 6-line) given by Table I, be drawn, it is seen that they gradually shift from *g h* to *a c*, and during the whole of this operation they appear to be tangents to a curve O *g*, *Fig. 14*. It would be interesting to find the equation to this curve, and also that of the 6-line derived from it. The latter is, of course, always on the outside of the "kerns" of the different trapezoids formed by the movement of *g h* round the curve O *g*.

The Paper is accompanied by a tracing, from which the Figures in the text have been prepared.

(*Paper No. 3609.*)

"Caisson-Disease at the New High-Level Bridge, Newcastle-on-Tyne."

By GEORGE WALTER MORGAN BOYCOTT.

THE explanation of the cause of caisson-disease which is now generally accepted is that first put forward by Professor Raneaux in 1861, and known as the "Air-emboli" theory. This theory was experimentally confirmed by the French physiologist, Mr. Paul Bert, in 1879; and others who have investigated the same subject are, on the Continent, Drs. Heller, Mager and Von Schrotter, whose results were published in 1898; and in Great Britain Drs. Leonard Hill and Macleod, whose results were published in 1903.

The conclusions arrived at by these experimenters may be briefly summarized as follows:—

(1) The blood-gases increase under pressure according to the law of Henry and Dalton.

(2) On rapid decompression, gas-bubbles escape, arresting circulation and causing pressure on the nerves and tissue.

(3) On re-compression, these gas-bubbles go again into solution.

(4) The varying gravity of the symptoms is due to the varying seat of the air-bubbles.

The law of Henry and Dalton is not, however, obeyed exactly, nor is it obeyed equally by all the gases present in the air. Time is required for the blood to become fully saturated; thus foremen, lock-tenders, and others whose duties necessitate their spending a comparatively short time under pressure, rarely suffer. Also, when the pressure is high, the number of cases of illness can be kept within reasonable limits by reducing the length of the shift. At Barmouth Viaduct, 1902, where the pressure reached was 46 lbs. per square inch above atmospheric pressure, the hours worked in each shift were 1½ hour at a time, twice a day. Under these conditions the cases of illness were, if anything, less severe and proportionately fewer than at Newcastle, where the pressure was 30 lbs. per square

inch and a nominal 12-hour shift, with two breaks, was worked. With regard to the increase of the blood-gases under pressure, Mr. Paul Bert found that at increased pressures the oxygen in the blood is increased only to a small extent; the carbonic acid is not increased to any appreciable extent; and the nitrogen is increased more than either the oxygen or the carbonic acid. Drs. Leonard Hill and Macleod obtained similar results.

The beneficial effect of re-compression has long been known, and at the Forth Bridge men suffering from "bends" were in the habit of spending their Saturday afternoons and Sundays in the ordinary lock or working-chamber. At a tunnel under the Hudson River a special medical lock was introduced by Mr. Moir. In this lock cases were subjected to compressed air, which was then allowed to leak out slowly. As a result of the use of this lock, and also of a larger supply of air per man, the death-rate, which had been 25 per cent. per annum, was reduced to 1½ per cent. per annum. It is necessary after re-compression that the subsequent decompression should be extremely slow, 30 minutes to 1 hour being allowed for a pressure of 30 lbs. per square inch, in order that the surplus air-gases may escape without again forming into bubbles.

At Newcastle, the three caissons which were sunk for the foundations of the New High-Level Bridge were 113 feet in length from nose to nose of the cut-water, and 35 feet in width. The working-chamber was 9 feet in height, on an average, and access to it was obtained by means of three shafts, each of which was divided into two portions, one for men and the other for materials. The air was pumped into locks which were fixed at the top of the shaft, passed down the shafts and escaped under the cutting-edge. Escape-valves were provided for the purposes of ventilation, but were used only during the concreting of the working-chamber, since for the rest of the time the air found its way out freely under the cutting-edge. Air was pumped simultaneously into two shafts, of which the centre shaft was always one. The number of men continuously under pressure varied between thirty and thirty-six on each shift, and free air was delivered at the rate of about 1,400 cubic feet per man per hour. Two 12-hour shifts were worked, and in pressures up to 22 lbs. per square inch the men had only the usual ½ hour for breakfast and 1 hour for dinner allowed to men working outside. At pressures exceeding 22 lbs. per square inch, each meal-interval was increased by ½ hour, and in addition a very liberal allowance was made for locking-in and locking-out, so that the actual time spent under pressure each day was 9½ to 10 hours at the lower pressures and 8½ hours at the higher pressures. The maximum air-pressure

reached was a little more than 30 lbs. per square inch, and decompression was effected at the rate of 5 lbs. per square inch per minute. This rate was very strictly enforced. Locking-in was done at the same rate, but no definite rule was laid down and many of the men were in the habit of going in very rapidly. For persons who can easily inflate their Eustachian tubes, or who are not suffering from a cold and who do not for other reasons experience any pain or inconvenience during the locking-in process, there is no danger in this being rapid. All men had to be examined by the doctor before starting to work under air-pressure, and any men found to be unsound were rejected.

It has been very generally held that the number of cases of illness varies as the percentage of carbonic acid present in the air. On the other hand, experiments on animals have shown that the increase of carbonic acid in solution in the blood under pressure is very small. It was therefore decided at Newcastle to take daily readings, during the sinking of one of the caissons, of the percentage of carbonic acid present in the air of the working-chamber, in order to ascertain whether any relation existed between the number of cases of illness and the percentage of carbonic acid. Samples of air were taken daily by the Author, one sample being taken a few feet south of the centre shaft and the other near that one of the two end shafts down which no air was being pumped. The percentage of carbonic acid present in each sample was obtained by means of the Haldane carbonic-acid apparatus. The fluctuations in the percentage of carbonic acid present in the samples taken near the centre shaft are shown in the diagram (*Fig. 1*). The percentage present in the samples taken near the end shaft was usually slightly higher. The high peaks in the first and third weeks were due to the air-engine not pumping to its full capacity. During the concreting of the working-chamber, on account of the free lime present in the cement, the percentage of carbonic acid fell rather below the point to which it fell during the week-ends when no work was being done. In order to supplement the more serious cases which would be reported to the doctor, and which would be too few to give any satisfactory results, a book was kept in which the foremen were requested to enter any cases, however trivial, which might come to their notice. These cases have been plotted in *Fig. 1;* a case is shown by a hatched square on the day on which it occurred. Thus on 27th October there were five cases. It will be seen that the cases are fairly evenly distributed, and that there is no marked increase when the percentage of carbonic acid is high. Indeed, the most serious cases, one in which a man was paralyzed, and another in which a

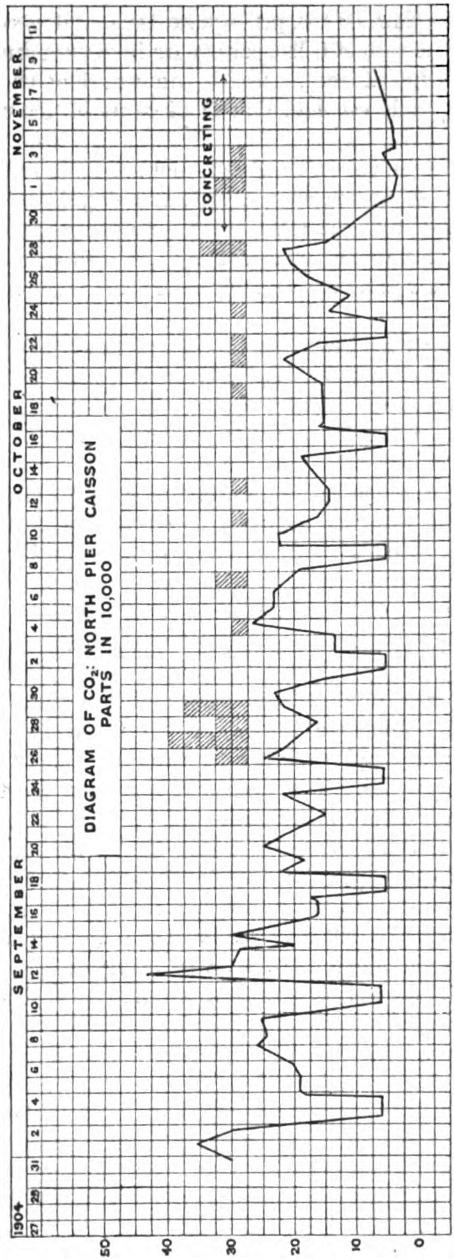


Fig. 1.

man lost 5½ shifts, occurred on 22nd and 24th October respectively, when the percentages of carbonic acid were rather below the normal. Of the cases plotted, four only were of sufficient gravity to come under official medical supervision.

On account of the form in which these figures are given, the Author has not been able to compare altogether accurately Dr. Snell's cases at the Blackwall Tunnel, taken from his book on compressed-air illness, with those at Newcastle. The results of such comparisons as he has been able to make, however, show that the cases of illness at Newcastle were neither more numerous nor of greater severity than those at the Blackwall Tunnel. The carbonic acid present in the working-chamber at Newcastle, as will be seen from the diagram (*Fig. 1*), was on an average about twenty parts in ten thousand, whilst at

the Blackwall Tunnel it was never allowed to rise above ten parts in ten thousand. The Author is of opinion, as the result of his observations at Newcastle, that the importance of good ventilation has been much over-estimated, and that the true preventive treatment of caisson-illness is slow decompression. The locks at Newcastle, although roomy and convenient as compared with many types with which the Author has had experience, were yet not large enough to render a long stay in them other than exceedingly irksome. He is of opinion that on work on which any large number of men is likely to be employed under air-pressure, special roomy locks should be provided in which decompression might be carried on at some such rates as those suggested by Drs. Leonard Hill and Macleod, which are given in the Appendix. When this is impossible the rate of decompression should be made as low as practicable, and the men should be encouraged to rest for a while before going home at the end of the shift, in order that if attacked by "bends" they may be treated by re-compression in the medical lock. Serious cases of illness generally come on within ½ hour of leaving the air-lock. Less serious cases may come on some hours afterwards. In many cases at Newcastle men have gone home and straight to bed, to wake up later suffering from "bends." The Author noticed that cases rarely occurred during a meal-time but nearly always at the end of a shift. This is probably because the effect of the three periods of the shift is cumulative; that is, the gas absorbed during a portion of a shift is not entirely liberated during the 1 hour or 1½ hour allowed for meals. It may also be that towards the end of the day the muscles are tired and therefore less elastic than at the commencement of work; and under these circumstances the liberation of the gas would be more difficult. This has been given as the reason why quite young men or youths enjoy greater immunity from "bends" than older men. The men employed at Newcastle were all comparatively young, the age of the oldest being about 42 years. There was therefore not much opportunity of observing that increased liability to illness which is generally held to exist amongst older men. With regard to another opinion generally held, that plethoric people are specially liable to illness, it may be mentioned that many of the men at Newcastle were stoutly built, and amongst these were two or three men whom the Author particularly noted as being peculiarly free from any form of caisson-illness.

Men who have worked under pressure for long periods appear to be as likely to suffer from "bends" as those who are new to the work. Of the two most serious cases which occurred at Newcastle, one was that of a man who had worked under pressure for the

contractors for many years. After working 3 or 4 weeks under pressure the men had a very sallow appearance but did not seem to be in any way unhealthy. Indeed, there was less ill-health among the caisson-men than amongst the men working in the open air on other parts of the works. At one time an outcry was raised by members of the firm's sick-club that caisson-men should pay an increased weekly contribution on the ground that they were specially liable to sickness. On investigation, however, it was found that the caisson-men were drawing proportionately less sick-pay than other members of the club; and at a regatta held amongst the men in the summer of 1904, during the active sinking of one of the caissons, more than half of the events were won by men who had been working under pressure right up to the day of the regatta.

A medical lock was provided at Newcastle. It was formed by converting an old air-lock which had been in use on a previous contract, and was fitted with a bunk, so that it accommodated one man lying down and another sitting. It was found to answer all requirements. The Author is of opinion that with slow decompression, such as is recommended by Drs. Leonard Hill and Macleod, proper medical supervision, and, when necessary, curative treatment in the medical lock, it would be possible, provided it were made compulsory for the men to rest after leaving the air-lock, to carry on compressed-air work with comparative immunity from illness at higher pressures than the 46 lbs. to 50 lbs. per square inch which has usually been considered the limit in this country.

The New High Level Bridge contract has been executed by Mr. F. W. Davis, Assoc. M. Inst. C.E., acting for the Contractors, the Cleveland Bridge and Engineering Company, Limited, assisted by Mr. C. R. S. Kirkpatrick, Assoc. M. Inst. C.E. The Author supervised the caisson-work under their direction.

The Paper is accompanied by a diagram, from which the Figure in the text has been prepared; and by the following Appendix.

[APPENDIX.

APPENDIX.

TABLE OF LENGTHS OF SHIFTS AND DECOMPRESSION-PERIODS SUGGESTED AS SAFE BY DRS. LEONARD HILL AND MACLEOD.

Lbs. per Square Inch above Atmosphere.	Shift.	Decompression-Period.
30	4 hours	30 minutes to 1 hour
45–60	4 hours	1 to 2 hours
75	1 hour	1 to 2 hours
90–105	30 minutes to 1 hour	2 hours

(*Paper No. 3620.*)

"On Tramway Permanent-Way Construction."

By Alexander Paterson, Jun.

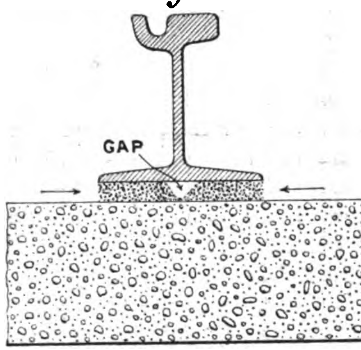
Having regard to the enormous amount of capital which has been invested in recent years in the construction and equipment of electric tramways, it is unnecessary to emphasize the great importance of considering carefully the construction of the permanent-way, upon which the life of a tramway principally depends, and the methods to be adopted in laying the track. In this Paper the Author directs attention to certain details in respect of (1) packing of rails, (2) rail-joints, and (3) paving, which contribute largely to the provision of a sound track, and to reduction of the cost of maintenance.

Packing of Rails.

One method of laying the rails, which has been extensively employed, is to excavate the roadway, put in the concrete, and, after it has been finished off smooth to the required level, to lay the rails on it and lift them up about ¾ inch by means of wedges or small stones; when the track is lined up and levelled off, a mixture of granite chippings and cement is forced into the ¾-inch space below the rails by means of beater-picks. The chief objection to this method is that, the packing-material having to be forced in from both sides of the rail-flange, it is impossible to ensure that no gaps are left under the web of the rail, as shown in *Fig. 1.* Should these occur, they eventually become filled with water, through condensation, sweating, or leakage, and every car passing over the rail forces the water into the packing; the car having passed on, the rail assumes its original level and the water returns to the gap, bringing with it particles of the packing. The continued repetition of this action ultimately destroys the packing, the setts alongside the rail begin to lift, and reconstruction of that portion of the line becomes necessary. An equally important objection to this method is due to the bottom of the rail-flange being raised above the surface of the concrete, thus pro-

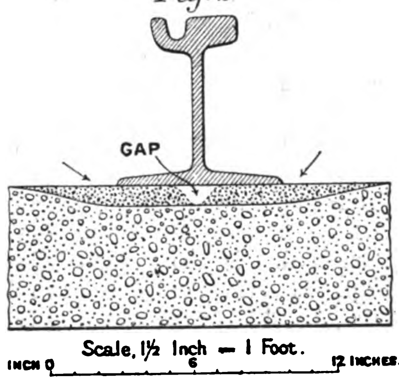
viding a reservoir between the rails for any surface-water which may get down through defective joints in the paving, or alongside the rail. The depression of the paving by heavy vehicular traffic forces this water to the low parts of the track; the water takes with it some of the sand bedding, and the removal of this soon causes the paving to sink and necessitates lifting and relaying. This objection might, of course, be removed by floating the track up to flange-level, and this would have to be done if the track had to be paved with wood; but the engineer has then to face the extra expense of, say, 2½ square yards of floating to every yard of single line, which at the low cost of 1*s.* per square yard amounts to £220 per mile.

Fig: 1.



The method of packing rails which is now more generally adopted is simply a modification of that just described. A hollow trench, about ¾ inch in depth, is left in the surface of the finished concrete, as shown in *Fig. 2*, and the rail is laid so that the bottom of the flange is level with the finished surface of the concrete, the packing-material being forced into the space under the rail by means of beater-picks. The advantage of this method is that the formation of a reservoir between the rails is avoided, but, on the other hand, it is obvious that the difficulty of insuring that no gaps are left in the packing-material under the rail-flange is increased, since the beater-pick cannot be so freely applied. There is also difficulty in making the hollow trench of uniform depth, and if it be not of uniform depth the thickness of the packing must vary.

Fig: 2



A further objection which has been raised against both of the foregoing methods is that a separate thin layer of concrete is being

put upon existing concrete which has already set. This may not be of much importance, but in the Author's opinion a system which would permit of the packing-material being part of and incorporate with the concrete bedding would be decidedly preferable.

Having adversely criticised the methods of packing rails almost universally adopted, it is incumbent on the Author to suggest an improved method, and the following mode of procedure is put forward as having proved, after several years of trial and testing, not only most efficient, but also most economical and expeditious. The excavation being completed, the rails are laid to the required levels, being packed up on old setts in preference to bricks or concrete blocks. These setts are left permanently in the work, embedded in the concrete; their removal after the concrete is put in would involve unnecessary expense, and would also necessitate disturbance of the concrete under the rail and the introduction of a separate body of concrete of no considerable bulk. The laying of the rails before the concrete bed is put in is an aid to the plate-layer rather than a hindrance, as it affords him greater latitude in adjusting the level of the rail-heads to the many variations of road-level than he would have were the concrete already laid. The setts supporting the rails are "staggered," and are spaced as far apart as possible without introducing the slightest tendency of the rails to sag. The concrete gang should not be allowed to rest their mixing-boards on the rails; with little trouble they can be placed on low trestles clear of the rail-heads. The plate-laying finished, the concrete is put in and shovel-packed under the rails as well as possible, the finer portions of the concrete being brought immediately under the rail-flange. The finished face should be slightly higher, say $\frac{1}{4}$ inch higher, than the bottom of the flange, in order to give a slight head to the concrete under the rail. This having been carefully done, a little time is allowed for the concrete to dry slightly before the final packing of the rails is commenced. If the packers follow the concrete gang at such a distance that there are always two clear rail-lengths between the two gangs, it will be found that a satisfactory result is obtained. The packing-material consists of clean $\frac{3}{4}$-inch granite chippings mixed with cement, almost dry, in the ratio of 3 to 1. The whole length of the rail is banked up with this material, about one-eighth of an ordinary shovelful being laid on each side of the flange per foot of single rail; and this is beaten until the whole is forced under the rail-flange, and then smoothed off. When properly done, hundreds of trials have failed to detect the least evidence of a layer; the packing-material becomes part and parcel of the concrete bed, and there is not the slightest sign of the concrete having suffered by the operation.

The precautions to be observed in packing rails by this method are:—

1. The chippings must not be too large or too small, ½ inch being the minimum and ¾ inch the maximum size; the object is to give vigour to the finer portions of the concrete bedding, which have been brought to the surface immediately under the rail-flange by the shovel-packing, and to avoid a honeycombed surface.

2. The mixture must not be too wet; otherwise it will shrink from the rail when setting.

3. For the same reason the packers must not approach nearer to the concreters than one rail-length and one-half.

4. They must not be more than two clear rail-lengths behind the concrete gang, as the setting of the concrete will then have gone too far to permit of its being disturbed without damage. If this limit be exceeded, inspection will reveal evidences of a layer, showing that the packing-material has not become incorporated with the concrete of the bed, and where the bed has been disturbed it will somewhat resemble frosted concrete.

The class of labour available for packing-operations is not the most painstaking, and, in the hurry and rush towards completion, defective work will occur in spite of the most careful supervision, and it is probable that upon few systems of any size has the test of traffic revealed no working-rail. It is therefore important to ascertain the best means of remedying such a defect. Undoubtedly it is best to start *de novo* and relay the defective portion of the track, but this, of course, involves stopping the traffic, or putting down temporary road and turning the car-traffic off the particular length until the concrete and packing have had time to set thoroughly. The best method of packing a moving-rail, while allowing car-traffic to pass over it, is a moot point, but the following method has, time after time, proved most efficacious. The concrete is well cleaned out to a depth of an inch below the rail-flange, and for 2 inches or 2½ inches on each side of it, and clean ¾-inch granite chippings are thoroughly beater-packed underneath it. The chippings should be cubical, free from dust, and such as will best stand the blow of the beater-pick without crumbling; they need not be packed too close together, but should be strong enough to support the rail, leaving sufficient interstices. These interstices must be filled, or water will collect in them and the remedy will be only temporary. Strong cement-grout with a little head would easily fill them, but the vibration of frequent car-traffic prevents it setting properly, and it is doubtful if it eventually becomes anything better than sand-grout. Pitch-grout, carefully tempered and poured in hot, has proved the best means of filling

these spaces, but great care must be exercised to ensure that there be not too much oil in the pitch, otherwise trouble will be experienced with lifting-setts alongside the rail, and, on the other hand, the pitch must be very hot, or it will not be fluid enough to fill all the interstices.

It may not be out of place to refer briefly here to a proposition which the Author believes is receiving the consideration of the authorities concerned with the tramway-system of a large city in the North where considerable trouble has been experienced in the maintenance of the permanent-way. The proposal is to lay the ordinary girder-rails on timber throughout, somewhat after the plan adopted at Hull, where, however, a rail having the groove in the centre is employed, and the ends of the rails are scarfed. It appears to the Author that, whether the timbers used be of the same width as the rail-flange, as in *Fig. 3*, or of a scantling wider than the flange as in *Fig. 4*, the weight of the car passing over the rail will compress the timber, and after the weight is removed the timber will return to its normal condition; this action will cause air-cracks

Fig: 3.

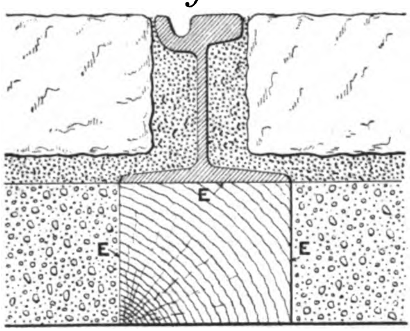
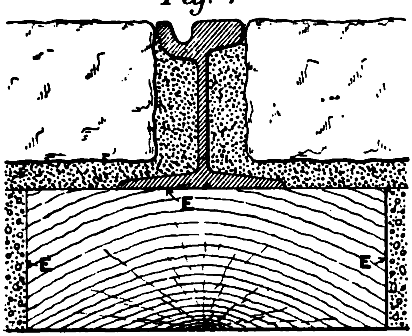


Fig: 4



Scale, 1½ Inch = 1 Foot.
INCH 0 6 12 INCHES

to be developed at E E E, *Figs. 3* and *4*, and it will be only a question of time before the whole of the paving alongside the rail will be on the move. In the Author's opinion, the arrangement shown in *Fig. 3* will be found preferable to that shown in *Fig. 4*, for the reason that a smaller area of paving will be affected by the moving timber. The problem to be decided in connection with this proposal is whether the saving in wear and tear of the rolling-stock would compensate for the increase in the cost of maintenance, which probably would be considerable,

Rail-Joints.

The subject of packing of rails leads naturally to a consideration of the various forms of rail-joint, as it is at the joints in the rails that failure of the packing most frequently reveals itself, and that most difficulty is experienced in doing good work. Starting with the ideal —a continuous jointless rail—it would seem that a close approximation to this might be secured by welding the ends of the rails together, provided that the process adopted were practical, not deleterious to the rail, and not too costly.

There are at present, so far as the Author is aware, four methods of welding rail-joints in use, namely (1) cast-welding, (2) electric welding, (3) "thermit" welding, and (4) the "weldite" process.

Cast-Welding.—In cast-welding, the rail-ends, having been cleaned, are surrounded by an iron mould, into which is poured a molten mixture of scrap-iron and soft and hard silicon-pig. Rightly or wrongly there is always a doubt as to whether a real weld is secured, which is supported by the fact that electrical engineers hesitate to accept the joint as affording a satisfactory electric bond. It is costly, is not easily adapted for repairs, and requires a somewhat cumbersome plant.

Electric Welding.—By this method the rails themselves are not welded. The ends, having been cleaned by means of an emery-wheel, are connected by one or two plates, somewhat resembling fish-plates; these are heated by an electric current passed through them, pressure is brought to bear, and they are thus welded to the rails. Expense, the cumbersome plant required, and the difficulty of obtaining the necessary current, especially in new lines, are serious obstacles to this method becoming universally adopted.

"Thermit" Welding and "Weldite."—For brevity, these two processes will be considered together; the mode of procedure in each is similar, and the results appear to be identical. The rail-ends are brought as close together as possible, and are surrounded with a small mould, over which is placed a furnace containing oxide of iron mixed with a small quantity of "thermit" or "weldite." The mixture is ignited, and rapid oxidation is set up, a very high temperature being attained. The molten metal is then allowed to flow into the mould, and the rail-ends are literally melted together. The root-principle of both processes is the remarkable and violent affinity which aluminium has for oxygen.

The objections to all these welding-methods are that they cost more than £1 per joint; that they increase the cost of plate-laying,

and retard the completion of the work; and that there appears to be considerable danger of the composition of the rails being altered, if not immediately at the rail-ends, at least where the red heat dies out.

Continuous Fish-plate Joint.—The joint which commends itself to the Author as the best is the "continuous fish-plate" joint. This joint is made with fish-plates rolled to the particular rail-section adopted, as shown in *Fig. 5*. They are rolled so that the points EE (*Fig. 5*) have to be "sprung" slightly in forcing them on to the rail-flange, and therefore grip the flange with a sort of wedge-action. This joint entails very little increase in the cost of plate-laying; the two rails are virtually one, since any slight differences in level of the rail-heads can be removed by filing; the composition of the rails is unaffected; and the cost is 50 per cent. less than that of the joints previously described. Careful inspection of a track, 20 miles in length, subject to a frequent car-service for more than 4 years, has failed to detect a single working-joint.

Fig. 5.

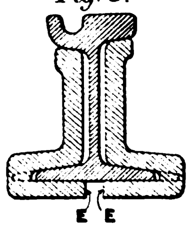
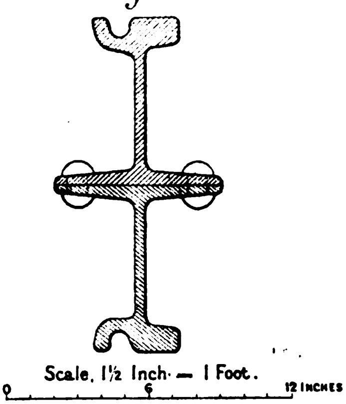


Fig: 6.



"Anchor" Joint.—A rail-joint which has been frequently adopted in recent years is the "anchor" joint. It is made by bolting or riveting a short piece of rail, in an inverted position, to the flanges of the two rail-ends, *Fig. 6*. This joint is intended to afford a rigid bearing to the two rail-ends, and to tie them down to the concrete so that there can be no lifting. There are two fatal objections to it. The portion of the rail which is supported by the anchor is more rigid than the remainder, and is therefore not so much depressed by a car passing over it; consequently, when the car passes from the joint to a portion of rail which is unsupported by the anchor, there must be hammering; in other words, the effect of the anchor-joint is simply to cause the hammering to take place further along the rail, instead

of at the end. Again, the microscopic depression of the rail at the joint is conveyed to the anchor, which is thus forced down into the concrete; and when the car leaves the joint the pressure is released, and an air-gap is left. The air-gap becomes filled with water, and, as car after car passes over the anchor, the head of the inverted rail becomes literally the plunger of a pump, causing destruction of the packing, bedding, and paving at the joint.

Paving.

However perfectly the rails may be bedded, and however smooth the joints may be, if the paving alongside the rail be carelessly laid all precautions to minimize the enormous annual cost of maintenance of permanent-way will be futile. It is therefore important to consider carefully the method of paving to be adopted.

All paving worthy of serious consideration for tramway-work is included under the two heads (1) granite-paving, and (2) wood-paving.

Granite-Paving.—Assuming that a perfectly-bedded rail and a smooth-running joint have been secured, the sources of expense in maintenance of the permanent-way are reduced to (1) vibration of the rail, and (2) wear, neither of which it is possible to remove entirely; it only remains, therefore, to take steps to minimize their effect as much as possible.

It is manifest that with heavy cars and frequent services the vibration in the rail-head must be very considerable and continuous. The vibration in the rail-flange, although perhaps less, is still also considerable, so that any sett alongside the rail has two of its surfaces exposed to a continually vibrating body, viz., the end of the sett against the rail and the bottom of the sett immediately over the flange. It is desirable to keep the sett from vibrating, if possible, and steps must therefore be taken to introduce between it and the rail some medium which, while holding it in position, will counteract or absorb vibration; the medium used at the end of the sett must be capable also of resisting percolation of water, whilst for the bedding-material between the sett and the rail-flange this quality is not essential.

For the bedding, sand is undoubtedly the best and cheapest material, but care must be taken to have the setts of fairly uniform depth, otherwise the weight of the vehicular traffic will cause some to sink more than others; the surface of the paving will thus be rendered uneven, and the effect of shaking of the higher setts by

impact of the wheel-traffic will be added to that due to vibration. The depth of the setts is, of course, determined by the depth of the rail, but experience shows that advantage can be gained by paying greater attention than is customary to this matter, since too great latitude in the depth of the setts gives rise to excessive vibration in the bedding; $\frac{3}{4}$-inch thickness of bedding, when the paving is thoroughly rammed, appears to be sufficient to minimize the effect of vibration. In recent years there has been a tendency to introduce cement into the bedding, but this, in the Author's opinion, is to be deprecated, for if sufficient cement is introduced to cause the bedding to set, the latter becomes a solid rigid body and loses its power to neutralize vibration, whereas if too little is put in it is a waste of material.

The selection of the medium to hold the sett in position, keep out water, and minimize the vibration transmitted to the end of the sett requires special consideration. If the foregoing assumption with regard to the effect of the addition of cement to the sand-bedding be correct, it is evident that the customary practice of filling the hollow between the sett and the rail-web with cement-grout or sand and cement cannot be recommended; but there is a certain elasticity in cement-concrete, and it would therefore to some extent deaden vibration. The advantage of using properly tempered pitch-grout as a means to counteract this movement must, however, be apparent. If good clean pitch be boiled with creosoted oil in the proper proportions, the product is a ductile, adhesive, watertight substance, which, at a temperature of 60° F. somewhat resembles india-rubber. Its ductility is probably due to the volatile gases of the oil being held imprisoned in the interstices of the molecules of the melted pitch. These imprisoned gases admittedly evaporate in time, but the process is slow, and the mixture will retain the necessary ductility for as long a period as the setts will wear. The following essential conditions must be kept in view in preparing the pitch-grout:—the pitch must be clean, and, like the oil, free from water, otherwise the mixture will boil over and splutter; there must not be too much oil in the mixture, otherwise the resultant grout will be too fluid and will never assume the desired india-rubber-like character; and, on the other hand, the proportion of oil must not be too small, otherwise the quantity of volatile gases imprisoned will be insufficient and the resultant grout will be brittle. It is difficult, if not impossible, to give the proportions in which the pitch and oil ought to be mixed, as the constituents of the pitch and the volatile gases of the oil vary so much. It becomes therefore a question of trial and error, and it is desirable that every cauldron should be tested before the grout is

used. If a small quantity is poured into water, in a few moments it can be taken out and tested at a temperature of 60 degrees; should it prove to be too brittle, more oil may be added to the cauldron before it is allowed to be used, and should it not set properly more pitch may be added. This primitive test would amply repay the small loss of time in making it, by increasing the probability of good properly-grouted paving being obtained. In grouting it is desirable that the racking in the joints of the paving be not too plentiful, and it would appear that pea-gravel is preferable to chippings, because the more or less spherical shape of the particles allows the grout to fill up the interstices more easily before the pitch sets. It is essential that the grout be poured in as hot as possible, and the setts should be dry before the grouting-operation is commenced, as pitch-grout does not adhere properly to wet setts. When pitch is poured right to the top of the setts, trouble is caused in hot weather by the pitch melting and running into the grooves of the rails. This can be avoided by keeping the pitch ½ inch from the top of the setts and brushing it over with a very wet mixture of sand and cement, gauged, say, in the ratio of 4 to 1. It is doubtful whether brushing over with sand-grout would not be as advantageous.

The question of the kind of granite paving to be used is outside the province of this Paper, but durability and absence of slipperiness are the chief desiderata. It appears to the Author that a 3-inch sett gives a better foothold to horses than a 4-inch sett, owing to the greater frequency of the joints. The joints should also be as tight as possible, as wide-jointed setts are manifestly more liable to be worn round by the horses' shoes than those which are tight-jointed. It is unfortunate that the most durable setts are also the most slippery, but as the greatest wear, caused by vehicular traffic, occurs alongside the rail, it would appear that a close-grained, hard, admittedly slippery sett might with advantage be toothed in alongside each rail, and the remainder of the track paved with the softer rough-grained sett, as the horse of his own accord, when avoiding the rail, keeps off the greater portion, if not the whole of the adjoining sett.

Wood-paving.—The behaviour of wood-paving is often mysterious. Not only have many tracks been put out of gauge by the swelling of the wood-blocks, but it is also found that, whilst one length of paving may lift, another length, paved with the same kind of blocks by the same men and in the same manner, may remain perfect. In laying the paving it is very important that the floating be true, and of first-class quality. The swelling of wood-blocks appears to take place

chiefly in the direction of their length, the increase in width being little or nothing. They ought, therefore, to be laid with tight joints, except at the ends, where the joints should be at least $\frac{1}{8}$ inch in width, and the two joints next the rail should be at least $\frac{3}{8}$ inch in width. The tighter the joints across the track the less possibility is there of the horses' shoes splitting the blocks. No advantage is to be gained by using blocks deeper than 4 inches; 1 inch thickness of concrete is manifestly cheaper than 1 inch thickness of wood. It is impossible to "up-end" the blocks and repave, and 1 inch wear off them would put the paving considerably below the level of the rail-heads. It is not proposed to discuss the relative merits of hard-wood and soft-wood paving, but where soft wood is used a row of hard-wood blocks might with advantage be laid alongside each rail to take the wear of the extra wheel-traffic upon this particular portion of the track; and if the tie-bars be cranked sufficiently to allow of the top edge of the tie-bar being level with the floating, the rabbeting of the blocks is done away with, and the formation of ruts in the paving where the tie-bars occur is rendered impossible.

The Paper is accompanied by three tracings, from which the Figures in the text have been prepared.

(Abstract of Paper No. 3598.)[1]

"Allowable Pressures on Deep Foundations."

By Elmer Lawrence Corthell, D.Sc., M. Inst. C.E.

In the year 1902 the Author was chairman of a Board appointed by the Argentine Government to consider and report upon propositions for the construction of a Port at Rosario on the Parana River, 180 miles above the city of Buenos Ayres. The Board consisted of seven members, all but one of whom were civil engineers. One of the subjects to which the attention of the Board was directed was the allowable pressure upon the material carrying the deep foundations of the quay-walls which it was proposed to construct.

These foundations were to be carried down through the alluvions of the river, and were to rest upon the Tertiary sand of the Parana River bed at depths ranging between 49·2 feet and 71·1 feet below low-water level, the bed of the river being 21·3 feet below low-water level.

The members of the Board held very divergent opinions upon the question of the pressure which might safely be imposed on the foundations, the pressures recommended ranging between 2·7 tons[2] per square foot and 5 tons per square foot, whilst the contractor to whom the work was entrusted (Mr. Hersent, of Messrs. Hersent and Schneider) proposed to allow a pressure of 7·3 tons per square foot upon the foundations.

The Board finally decided upon a limit of 3·2 tons per square foot. The Author, however, was not satisfied with this decision, and, by communicating with the members of the Board individually, he found that a majority were of the opinion that 5 tons per square foot would be quite safe, and in the contract for the work as finally arranged this pressure of 5 tons per square foot was fixed upon as a maximum, with a mean of 3·34 tons per square foot.

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[1] The complete Paper, together with the Tables and voluminous Appendixes, may be seen in the Library of the Institution.

[2] Throughout this Abstract the word "ton" means 2,240 lbs. In the original Paper it is used to indicate the United States ton of 2,000 lbs.

The wide difference of opinion amongst experts, disclosed by this inquiry, caused the Author to make further investigations, of which this Paper is the result.

A circular letter was addressed to about three hundred engineers in various countries, asking them to contribute information on the subject, but only about thirty replies of any value were received. The Author was assisted in the inquiry by Mr. C. R. Wychoff, who was also instructed by the Author to make a thorough study of all published data on the subject and to collate the results.

The Tables which accompany this Paper contain data, more or less complete, in reference to one hundred and seventy-eight works. The deficient feature, as will be seen from an inspection of the Tables, is the dearth of information in reference to pressures upon the material. The Tables also, no doubt, require to be corrected in some particulars, and it has been difficult to ascertain whether the pressures are maximum or mean, and also whether they include buoyancy of the surrounding water and the frictional resistance of the sides of the construction.

The mass of information which the Author has collected on the subject is embodied in four Appendixes to the Paper, viz.:—

Appendix A, consisting of eight Tables, giving the results of the replies to the Author's circular letter of inquiry.

Appendix B, consisting of a report on the proposed Tampico Wharf, Mexico,[1] by Messrs. Tulloch and Alfred Noble.

Appendix C, containing some of the correspondence received and data collected.

Appendix D, containing the results of Mr. Wychoff's investigation.

The Author has made an analysis of the data given in the Tables which embody the replies to his circular letter, and he has also included the data given in the Tampico report. The result shows that the pressure of stable structures on the material upon which they are founded ranges between the following limits:—

On *fine sand*, between 4,500 lbs. and 11,600 lbs. per square foot, with an average of 9,000 lbs. per square foot in ten examples.

On *coarse sand and gravel*, between 4,800 lbs. and 15,500 lbs. per square foot, with an average of 10,200 lbs. per square foot in thirty-three examples.

On *sand and clay*, between 5,000 lbs. and 17,000 lbs. per square foot, with an average of 9,800 lbs. per square foot in ten examples.

On *alluvium and silt*, between 3,000 lbs. and 12,400 lbs. per square

[1] Since constructed, *see* Minutes of Proceedings Inst. C.E., vol. clxiv, p. 464.

foot, with an average of 5,800 lbs. per square foot in seven examples.

On *hard clay*, between 4,000 lbs. and 16,000 lbs. per square foot, with an average of 10,160 lbs. per square foot in sixteen examples.

On *hard pan*, between 6,000 lbs. and 24,000 lbs. per square foot, with an average of 17,400 lbs. per square foot in five examples.

These cases show no settlement. The range is considerable and no doubt in the case of the minimum pressure a much larger weight could have been imposed on the material without producing settlement. For a safe rule, therefore, the average is low and a safe pressure upon the material would lie somewhere between the average and the maximum pressure.

Besides the foregoing cases the Author found three instances in which notable settlement took place in fine sand, with a minimum load of 3,600 lbs. per square foot, a maximum load of 14,000 lbs. per square foot, and an average load of 10,400 lbs. per square foot. In clay, principally London clay, the Author found five examples in which the pressure ranged between 9,000 lbs. and 11,200 lbs. per square foot, with an average of 10,400 lbs. per square foot. In silt and alluvium two cases of settlement were reported, with loads of 3,200 lbs. per square foot and 15,200 lbs. per square foot—a wide variation. Three cases of failure on a mixture of sand and clay are given, the pressures ranging between 3,200 lbs. and 14,800 lbs. per square foot, with an average of 6,600 lbs. per square foot. This should be compared with the average of 9,800 lbs. per square foot and range of 5,000 lbs. to 17,000 lbs. per square foot in ten examples, already referred to, in which no settlement occurred.

Records of frictional resistance are also very variable. In ten cases of cylinder-piers the average was 540 lbs. per square foot, the resistances ranging between 300 lbs. and 1,500 lbs. per square foot, gravel showing the greatest resistance (1,500 lbs. per square foot) and mud the least. In twenty-three masonry-piers the frictional resistance ranged between 300 lbs. per square foot in sand and gravel, and 1,000 lbs. per square foot in sand and clay, with an average of 522 lbs. per square foot.

The Author concludes his Paper with a description of the foundations of the fallen Campanile of San Marco at Venice, showing the character of the soil and giving an estimate of the pressures which it sustained for 1,000 years. He also draws attention to an interesting Paper on foundations by Mr. J. R. Worcester, presented to the Boston (Mass., U.S.A.) Society of Civil Engineers on 28 January 1903.

(*Paper No. 3607.*)[1]

"The Construction of the Seaham Harbour Dock-Works."

By PERCY TILLSON GASK, M. Inst. C.E.

THE old docks at Seaham Harbour, built by the grandfather of the present Marquis of Londonderry about the year 1828, consisted of a North dock, 2¾ acres in area, having an entrance 33 feet in width, with a depth of water on the sill of 16 feet at high-water; a South dock of about the same size; and a tidal harbour of about 3 acres in extent. The sea-approach to these docks was protected by a North and a South pier, with an opening between them of 120 feet.

The piers, as well as a large portion of the dock-walls, were built with a facing of large squared blocks of masonry, and a hearting of rubble. They are all founded on the rock, and have stood the test of time remarkably well, although the lime-mortar used in some of the joints is as soft to-day as when it was put in. The stone was brought from Pensher, a quarry about 6 miles inland from Seaham; some of the blocks weigh as much as 7 tons, and must have entailed great labour in bringing them from the quarry by road and setting them without the aid of modern appliances.

The capital outlay on these works appears to have been about £120,000.

The coal for shipment is brought from the Seaham pits, situated about 1½ mile inland, by means of a self-acting incline terminating on the staithes themselves, from which the coal is discharged down shoots into the holds of vessels lying alongside. The shipments in the past have been, on an average, about 600,000 tons per annum, and have risen as high as 800,000 tons per annum. The facilities for shipment were such that a vessel of 900 tons carrying-capacity could be loaded in less than 2 hours.

The advent of steamers of increased size and draught necessitated a considerable extension of the dock-accommodation; and as, about

[1] Paper read before the Newcastle-upon-Tyne Association of Students of the Institution, 23rd March, 1905.

the same time, it had been decided to sink in the neighbourhood three new pits, which from their position would naturally use Seaham Harbour as their port of shipment, it seemed evident that a satisfactory return would be obtained on the additional capital outlay.

The Seaham Harbour Dock Company was therefore formed in 1898, and took over from Lord Londonderry all his rights in the existing undertaking for a sum of £88,350, and obtained an Act of Parliament for the construction of a new dock and protective piers.

The contract was let to Messrs. S. Pearson and Sons, Limited, for the sum of £378,000, and the preliminary works were begun in March 1899. The new works comprise two protective piers enclosing the site of the old docks, as well as a new 10-acre dock, 1,000 feet in length by 450 feet in width, having a depth of 27 feet 6 inches below high-water of spring-tides, Fig. 1, Plate 6. The entrance to the new dock is 65 feet in width, and the sill is 2 feet above the level of the dock-bottom. The entrance is closed by a pair of timber gates, and for use in rough weather storm-gates are provided. The channel from the entrance to deep water is to be dredged to a depth of 24 feet 6 inches below high-water level.

The first work proceeded with was the reclamation of land on the north side to form a blockyard for the construction of the concrete blocks of which the two piers were to be built. This was done by running out from the shore a timber gantry, from which a row of whole-timber sheet-piling was driven, and the space thus enclosed was filled up and levelled off. An area of about 2½ acres was thus obtained, and on this the blockyard was laid out. The plant provided consisted of two Taylor continuous-running concrete-mixers, to which the ballast was fed by endless chains of buckets, the cement being hoisted up by suitable gear from a storage-shed built behind the mixers. From the mixers the concrete was run by hand in small tip-wagons on to low staging commanding the various block-moulds. The ballast for the mixer was obtained from the sea-shore, and contained enough sand to be used without further addition of that material. The blocks are formed of 6-to-1 concrete and those forming the outer surface of the piers are faced with stone, obtained partly from the Lartington Quarries near Barnard Castle and partly from the old dock-walls. The moulds were struck 2 days after they were filled, and the blocks were stacked 5 days later. Only one mixer was used at a time, the other being kept as a stand-by in case of breakdown, and the output for a full day's work from one mixer was about 200 cubic yards. For lifting and stacking the blocks, a 30-ton "Goliath" steam-crane was used, having a span of 100 feet between the centres of the two bogie-roads.

The only connection between the north and south sides of the old docks was by means of a rope-incline to the top of the cliffs, which at this point are 80 feet above sea-level, and down again by another rope-incline. To avoid taking the blocks for the south pier this way, a tunnel was cut through at quay-level and has been of great assistance in handling the materials.

North Pier.—The two root-walls of the north pier enclose the mass of rock upon which stands the old lighthouse, and under which a huge cave had been formed by the action of the sea, Fig. 1, Plate 6. These walls were built of 4-ton blocks, from a staging running alongside, and, as the foundations for them were bare at low-water, this part of the work was done without much difficulty. From the junction of the root-walls, the pier proper starts, and has a total length of 1,383 feet. It is built to a radius of 1,440 feet, and it was found by experience that special long blocks had to be provided at intervals along the outer face in order to keep the scar-end radial. The inner half of the pier has a width at coping-level of 25 feet, and the outer half is widened to 30 feet as the depth of water increases and the gradual curvature of the pier causes it to be more directly exposed to the north-east, from which direction the heaviest seas are experienced. A flight of granite steps is formed where the change of width is made. The surface of the pier is 9 feet above high-water level, and the sides are formed to a batter of 1 in 11, with two offsets of 1 foot each on the foundation-courses, Fig. 2, Plate 6. The blocks used vary between 12 tons and 28 tons in weight, and are arranged as far as possible in Flemish bond, in courses 5 feet 6 inches in depth. The larger blocks are 12 feet in length by 6 feet in width. Below low-water level they are set dry and have half-round joggle-holes, 9 inches in diameter, formed in the sides, into which, after the blocks are in position, specially-made bags of concrete are dropped, to prevent lateral movement. Above low-water level the blocks are set in 2-to-1 cement-mortar.

For setting these blocks a Titan crane, capable of lifting 30 tons at a radius of 60 feet, was obtained by the contractors from Messrs. Stothert and Pitt at a cost of about £6,000. It was so constructed as to be capable of slewing through a complete circle, and also to allow of the passage of wagons between the two legs. Subsequently an air-compressing plant for working the diving-bell and a dynamo for lighting at night were fixed on the engine-platform. The reach of the crane governed the length of pier which could be built at one time, and bays of 24 feet in length were decided upon.

The foundations of the pier are on the magnesian limestone formation throughout, and vary in level between about low-water

level at the shore end, and 17 feet below that level at the outer end.

In places the rock is covered by a layer of sand, up to 6 feet or 8 feet in thickness, but the surface of the rock was of a very uneven character, large masses standing up 6 or 8 feet above the surrounding level, and in other places deep gullies running across the line of the pier. Under these circumstances it was decided not to dress down the surface of the rock for the reception of the blocks, but to level up the hollows with mass-concrete deposited by divers from skips under water. This concrete is extended 8 feet to 10 feet outwards on either side of the pier to form an apron, to prevent undermining of the blockwork. The concrete used for this work was gauged 4 to 1; the surface was screeded off by divers at the correct level for the commencement of the blockwork, and this level was raised or lowered by the depth of a block in order to follow the general contour of the rock. This type of foundation necessarily entailed considerable waste of concrete, for, if the sea happened to rise before the concrete had had time to set, it was either washed away entirely or nothing but clean ballast was left, which had to be removed before operations could be resumed. In some cases during the winter months a bed would be screeded three times before blocks could be set on it. In good weather two divers could prepare a bed 24 feet in length and the full width of the pier in 2 days.

Where the rock was covered with sand, the latter was removed as far as possible by a Hone grab of 6 cubic yards capacity, worked from the Titan crane, and the material which could not be removed in this way was cleaned up from a diving-bell. An attempt made to scour away the sand with a water-jet proved unsuccessful, owing partly to suitable plant not being available. When the foundations had been prepared, little time was required for the setting of the blocks, given favourable weather; the chief difficulty was caused by the washing-out of the joints by the rising tide, and to minimize this as much as possible neat cement mixed with a solution of soda was used for pointing the outer joints. The soda-mortar set very rapidly and protected the ordinary cement-mortar inside. It was afterwards cut out and the joints were then pointed in the usual way. A subway is formed in the blocks down the centre of the pier to provide an approach to the lighthouse at the pier-end in bad weather.

The number of days on which work of any kind was possible on the pier was 122 days in 1902; 140 days in 1903; and 168 days in 1904: and of these only about two-thirds in the first two years and nine-tenths in the third year, were really effective working days.

The lengths of the portions of the north pier built in the different years were 220 feet in 1901; 212 feet in 1902; 270 feet in 1903; and 390 feet in 1904: and the shortest time occupied in completing a bay 24 feet in length was 7 days, involving the handling of about 1,200 cubic yards of mass-concrete and blockwork, the work being carried out by a single gang of about fifteen men working a considerable amount of overtime. The longest time which elapsed between the completion of two successive bays was practically 6 months, viz., 28 October, 1904, to 27 April, 1905.

The outer end of the pier is finished off with a roundhead 40 feet in diameter, formed of blocks dove-tailed into one another, and on this a lighthouse will be built. The surface of the pier is to be paved with granolithic concrete between the copings.

The cost of the north pier amounts to about £90 per lineal foot.

South Pier.—The south pier is 878 feet in length, starting from root-walls at the junction of the south harbour-wall and the south sea-wall. It is similar in construction to the north pier, except that it is only 20 feet in width at the inner end and 25 feet in width at the outer end, and is built of 20-ton blocks in courses 4 feet 6 inches in depth, Fig. 3, Plate 6. It was not necessary for this pier to be as massive as the north pier, as the depth of water is less, and the pier is sheltered from the worst gales, which come from the north-east. The foundations are on rock throughout, but owing to their being only a few feet below low-water level even greater difficulty was experienced than in the case of the north pier, the shallowness of the water at low tide causing a greater scour. The opening between the two pier-ends for the passage of vessels is 280 feet. The entrance faces south-east, and the distance from it to the new dock-entrance is about 300 yards.

New Dock.—The new dock, which covers an area of 10 acres, absorbs the old South dock and dry harbour. In addition there are about 7 acres of quay-space to the south, the whole of which is on ground reclaimed from the sea. To exclude the water was therefore the first operation, and to do this the south sea-wall was started, and later on the south harbour-wall, from a point at the south-east corner of the old dry-harbour wall.

The south sea-wall is built of 6-to-1 mass-concrete deposited at low tide between temporary timber shuttering, Fig. 4, Plate 6. It starts from the cliff at the inner end, and runs out in a series of curves for a total length of 958 feet. Its height varies between 14 feet at the inner end and 22 feet at the outer end, and its thickness at the bottom is 12 feet. The finished wall contains 6,000 cubic

yards of concrete, and occupied 14 months in building. It presents no special features, and the only difficulty experienced in its construction was that due to its exposed position—no doubt a very real difficulty to the contractor, who often had to face the washing-away of a large amount of concrete and temporary staging as soon as it had been put in place.

The south harbour-wall, forming the eastern boundary of the area to be reclaimed, was built of 4-ton blocks faced on the outer side with masonry. It is 9 feet in width and 20 feet in height, on an average, and withstood in a most satisfactory manner the severe gales to which it was exposed before it had been backed up. Later on, when the south pier had been built out for a short distance and the action of the sea was concentrated in a corner, a breach was formed and considerable damage was done. Whilst the outer walls were in progress, excavation was going on inside the old dry harbour, from which the water had been excluded by a timber-and-puddle cofferdam across the entrance. The bottom had to be deepened by about 12 feet, and, except for a layer of silt on the surface, the whole of this depth was in the limestone rock. A Ruston and Proctor steam-navvy was used, and with the aid of blasting was able to deal with the rock satisfactorily. The hardness of the rock varied greatly; in some places thick beds of flinty hard rock were met with, which required considerable blasting, and in other places nothing but soft yellow marl had to be removed. Many springs were encountered, and a 12-inch rocker-pump and a 10-inch centrifugal pump were fully occupied in keeping down the water. During heavy seas the water broke over the outer walls and work was stopped. In March, 1901, when the excavation inside the dry-harbour walls was well advanced and the enclosing of the ground to the south had been completed, an exceptionally high tide, combined with a heavy sea breaking over the harbour wall, ponded up water in the enclosed area to such a height that a "run" was formed under the south wall of the old dry harbour, and eventually a section of the wall gave way. The water rushed in with great force, broke down a pair of connecting-gates leading into the old South dock, and caused an old tug-boat, which was moored to the quay-wall, to break adrift and to be swept out to sea, where it became a total wreck. Had the protecting-piers been nearly completed before the dock-work was commenced this failure might have been avoided, but the importance to the Dock Company of speedy completion of the new dock prevented this course being adopted. After this accident it was decided to discontinue the use of the old South dock for traffic purposes, and the dock was handed over to the contractors, who put a concrete dam

across the entrance, pulled down the south walls, and proceeded to excavate the bottom down to the new level.

The old north wall was to form the northern boundary of the new dock, and it was specified to be under-pinned so as to carry it down to the new dock-bottom, but on examination it was found to be so insecure, being merely a facing-wall to the natural rock, which showed through in many places, that it was decided to re-face it from top to bottom. To do this, 14-inch by 14-inch timbers, about 33 feet in length, were placed at intervals of 8 feet along the face of the wall, the lower ends being secured in holes cut in the rock, and the upper ends and the centres being tied back into the rock by 2-inch tie-rods. Shutters were then placed along the front of the timbers, and concrete was filled in against the face of the old walls. A facing of rather more than 2 feet thickness of concrete was thus obtained over the whole area, held in place by the vertical timbers and by numerous pockets or ties cut into the old wall or into the rock, Fig. 5, Plate 6. A coping completed the wall, and elm fenders were afterwards added to the faces of the vertical timbers.

The walls on the west and south sides of the new docks were built in trench below rock-level, and that on the east side was built after the excavation had been completed to dock-bottom. The type of wall adopted was that of an ordinary retaining-wall 10 feet in thickness at the bottom, resting on the rock, and below this a facing of concrete 4 feet 9 inches in thickness, with a toe 7 feet in width founded 1 foot below dock-bottom, the total length of the wall being 35 feet 6 inches, Fig. 6, Plate 6. The concrete was all gauged 6 to 1, and the upper 14 feet 6 inches was faced with masonry and projected 3 inches in front of the concrete face. A granite coping, 3 feet in width by 1 foot 6 inches in thickness, completed the wall at a height of 8 feet above high-water level.

The greater part of the concrete for these walls was mixed in a Carey and Latham revolving cylindrical drum-mixer, which could be moved along on wheels as the work proceeded. From the mixer the concrete was deposited in the walls by means of steam-cranes, and skips of 1 cubic yard capacity. Large stone displacers or plums were used in the wall to a limited extent. Weep-holes through the walls were left at intervals a short distance above rock-level; these were plugged up just before the water was let into the dock, but were subsequently re-opened and fitted with flap-valves, as it was found that the ground behind the walls became highly charged with water.

Failure of Dock-Wall.—The failure of a length of the east dock-wall took place in October, 1903. At that time the water had been admitted into the new dock, and it was being partially used for traffic as a

tidal basin, the new entrance not being complete. Of the north pier 480 feet had still to be built, and of the south pier 500 feet, so that the south harbour-wall was still exposed to a considerable extent to the north-east. On the 9th October the afternoon tide rose to 4 feet above ordinary high-water, and was accompanied by a very heavy sea, which broke in great volumes over the south harbour-wall and scooped out a certain amount of the filling behind the inner dock-wall. Suddenly, without any warning, and whilst the water in the dock was standing within 4 feet of coping-level, a length of about 450 feet of the east dock-wall fell bodily forward into the dock, and the filling behind was soon washed away to a flat slope, carrying with it plant and railway-lines. Subsequent investigation showed that the wall had broken off in the narrow part of the leg a few feet above the toe, and that more than 7,000 cubic yards of material had been carried into the dock, Fig. 7, Plate 6. As soon as possible after the accident the slope was roughly pitched with large blocks of stone from the old dock-walls, and the upper surface was concreted to prevent further damage. The rebuilding of this wall has not yet been commenced, but a large amount of the débris has been removed by dredging and by divers. The distance between the harbour-wall and the inner dock-wall is 125 feet, and the failure of the wall at extreme high tide is the more remarkable on account of the fact that before the water was allowed to rise in the dock the filling behind the wall was several times saturated to such an extent by seas breaking over the outer wall as practically to put a full hydraulic pressure on the back of the wall, as was shown by the force with which water spurted out from the weep-holes left in the wall. The only explanation that seems at all feasible is that either the shock of the seas breaking against the harbour-wall shook the inner dock-wall to such an extent as to break the seal between the lower part of the concrete and the rock, and so allowed a film of water to penetrate, or else the same effect was produced by water penetrating through fissures in the rock, which was afterwards put under excessive hydraulic pressure due to the weight of seas falling on the water already standing behind the wall where the filling had been washed out. That the weight of the filling did not exert any material pressure on the back of the wall was evident from the fact that it stood with practically a vertical face for some appreciable time after the wall fell, before it was worn away by the water running over it. It was never anticipated that this wall would be subjected to such a severe back-pressure, for if the pier-works had proceeded with anything like the speed originally contemplated the harbour-wall would have been protected before this dock-wall was built.

The bad seasons of 1902 and 1903 materially retarded the pier-work.

New Dock Entrance.—The site of the new dock entrance was crossed by the west wall of the old dry-harbour, and for a time this was used as a dam and as much of the work as could be done inside it was carried out whilst the excavation of the dock was in progress. An inner dam was then built round the portion already completed, in order that, as already mentioned, the dock might be partially used as a tidal harbour until the new entrance was completed and the piers were sufficiently advanced to protect it. For this purpose a temporary gap 45 feet in width was formed, which was sheltered by the old piers; and by means of a movable boom raised and lowered by winches a depth of 13 feet of water was maintained at low-water. An outer dam was then constructed round the sea side of the new entrance. This dam was 400 feet in length and about 24 feet in height, and was formed of concrete blocks 9 feet in width. In such an exposed position it was impossible at a reasonable cost to form a dam which would keep out the water in all weathers; the top of the dam was therefore kept about 8 feet above high-water level, and when rough weather came and the seas broke over, work inside had to be stopped. An 18-inch direct-driven centrifugal pump was put down to deal with as much of the storm-water as could not be let out through sluices, and the water which found its way through fissures in the rock was dealt with by means of one 10-inch and one 12-inch Evans pump. About 20 feet thickness of rock had to be removed over the area of the new dock in order to get down to foundation-level, and all springs met with were led in pipes to the sump and will eventually be grouted up. A layer of concrete, varying in thickness between 10 feet under the sill and 6 feet under the aprons, was then put in and the walls were proceeded with. The sill is faced with granite stones 4 feet 6 inches in depth, standing 2 feet above the gate-platforms, and the distance from apex to apex between the inner and the storm-gates is 44 feet. The distance in the clear between the walls is 65 feet and they are faced throughout with masonry and backed with 6-to-1 concrete. Granite hollow quoins are built in for the heel-posts of the gates, and culverts, 7 feet by 5 feet, closed by sluice-paddles, are formed in each wall, for controlling the water in the dock.

The two pairs of gates are made of karri-wood from Western Australia. This timber is of the blue gum tribe and has the advantage over greenheart that logs 2 feet square, for forming the heel- and mitre-posts and fishing-pieces, can be readily obtained out of one

piece, instead of being built up of several logs, as would be necessary if greenheart were employed. It is not affected by sea-worm to any extent in this part of the world, but it has the defect that when sawn up and exposed to the air, surface-cracks develop, sometimes to a serious extent, and it was found desirable, after the gates had been built in a shed and taken to pieces ready for erection, to bury the timbers under a covering of soil in order to keep them damp and away from the action of the sun. Each leaf of the gates when completed weighs about 90 tons. Strut-gates are provided for use in stormy weather to relieve the strain on the chains and to stiffen the gates. The two pairs of gates, together with strut-gates, cost about £10,000 in position. Hydraulic rams, placed in pits below coping-level, operate the gates by means of chains passing through passages formed in the side-walls. Two 5-ton hydraulic capstans are fixed on either side of the entrance for assisting in the passage of vessels and for working the strut-gates.

Coal-Staithes.—The staithes for the shipment of coal, erected along the west wall of the dock, are arranged to deal with high-capacity wagons, and, owing to the height of the cliffs upon which the storage-sidings will be placed, it has been possible, as at Tyne Dock and Sunderland, to dispense with the use of engines, and to do the work by gravity from the time the full wagons leave the storage-sidings until the empty wagons have left the staging. The spouts are carried on pitch-pine trestles, spaced 16 feet apart, upon which are placed longitudinal steel joists carrying the flooring-timbers and rails. The height of the rails, at the spouts, above the ordinary water-level in the dock varies between 51 feet and 54 feet. The old spouts along the north end of the dock will still be used. It is anticipated that when all the staithes have been brought into use a shipment of 2,000,000 tons of coal per annum will be dealt with.

The scheme was designed, under instructions from Lord Londonderry, by Mr. H. H. Wake, M. Inst. C.E., Engineer to the neighbouring port of Sunderland. On the formation of the Company Mr. P. W. Meik, M. Inst. C.E., Westminster, was appointed Engineer, and the works have been carried out under his supervision, the Author acting as Resident Engineer.

The Author desires to express his indebtedness to Mr. Thomas Thomson, M. Inst. C.E., of the firm of Messrs. S. Pearson and Sons, Limited, for information contained in this Paper.

The Paper is accompanied by two drawings, from which Plate 6 has been prepared; and by twenty-nine photographs, which may be seen in the Library of the Institution.

(*Paper No. 3618.*)

"Repairs to Dock Walls."

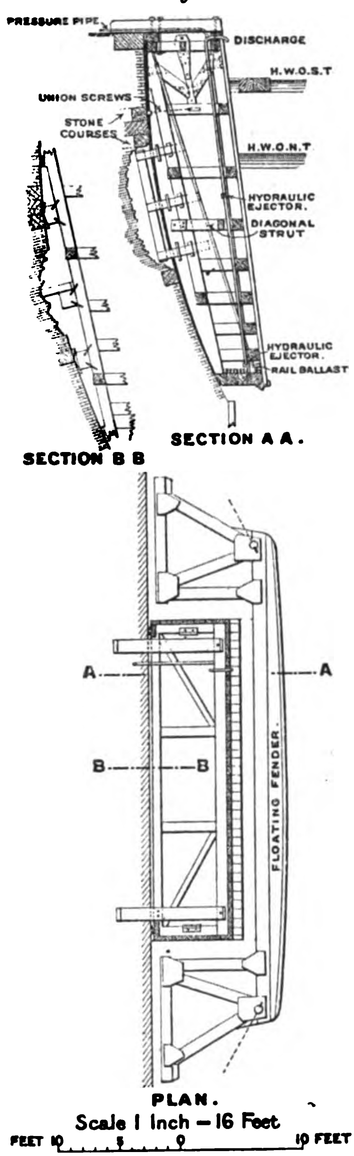
By Frederick James Rigg, Assoc. M. Inst. C.E.

The Humber dock at Hull, which was opened in 1809, was built with its walls battered on the face to a uniform radius of 66 feet, the sheet-piling at the toe being about 8 feet in front of the coping. This profile was no doubt well suited to vessels of a century ago, but modern steamers, almost rectangular in midship-section and fitted with fenders and rolling-chocks, chafed the face of the wall to such an extent that a general overhaul was decided upon.

To carry out the repairs below water-level, a timber caisson was constructed, as shown in *Figs. 1.* The transverse timbers shown transmitted the weight of the caisson to the wall, with which it was held in close contact by union-screws, one at each end, attached to lewis-bolts in the upper stone course. The water-tight joint at the wall was made with oakum, packed in canvas bags and laid in shallow grooves on the lower and two end faces of the caisson, the lower portion being covered with old grain-conveyor bands as a protection against abrasion by the wall when being placed in position. This proved sufficient where there were no breaks in the line of contact, but for a considerable length of the wall the gap measured 10 feet by 3 feet, and to fill this a special pad was made to a template prepared by the diver. The wooden slides shown in the section AA, *Figs. 1*, were pushed into the gap, and prevented the pad from being blown inwards by the outside pressure. Cinders, ashes and sawdust helped to improve the joint, and small leakages were kept under by two hydraulic ejectors in tandem in the caisson, coupled up to a temporary main laid along the quay-edge. For pumping the water from the caisson a tug's fire-pumps were employed, and as

the water-level was lowered the intermediate frames were strutted off the wall by driving in wedges at the distance-blocks as shown in the section BB, *Figs. 1.* The total pressure between the caisson and the wall at high-water of spring-tides amounted to about 260 tons. The water having been pumped out, the damaged portion of the wall was cut away to enable the new brickwork to be properly bonded into the old; this was somewhat difficult to effect, as the interior of the wall consisted of bricks varying between 1½ inch and 3 inches in thickness, lying at all angles. Common red bricks were used to fill up the gaps, and Staffordshire blue bricks for the face. Each successive position of the caisson was so arranged that one vertical joint was made on the last repaired face, thus leaving only the forward end and bottom to be made tight against the old wall.

Figs. 1.



The weight of the caisson, including ballast and allowing for buoyancy, was about 22 tons, and for moving it from one position to another two mud-lighters were fitted with lifting-tackle, which allowed of its being picked up and set down independently of the water-level in the dock. Steamers occupied their usual berths during the progress of the work, being

kept clear of the caisson by the floating fender shown in the plan, *Figs. 1*.

The dock is the property of the North Eastern Railway Company, and the work was carried out by the Dock Engineer's Department.

The Paper is accompanied by two drawings, from which *Figs. 1* have been prepared.

(*Paper No. 3622.*)

"Irrigation in the Transvaal."

By Michael Richard Collins, Assoc. M. Inst. C.E.

The object of the Author in this Paper is to indicate, on somewhat general lines, certain problems bearing on the development of irrigation in the Transvaal; to deduce the run-off, flood-discharge, and duty which appear to be applicable in the design of storage-reservoirs; to estimate the revenue which may be expected from such works; and to investigate the possibilities which the country presents with regard to settlement. A brief description is also given of some existing Government works.

The Law.—In order to appreciate the difficulties which attend irrigation enterprise in the Transvaal, it is necessary to consider the legal aspect of the subject. The law which at present controls the usage and rights with regard to water in that Colony is Law No. 11 of 1894. As its clauses are general and indefinite, and as it must be read in conjunction with the common law, and with previous court decisions, its interpretation has been entirely in the hands of the legal profession, and of the courts. It recognizes riparian rights, and the water in "public streams" is therefore utilized by individuals, and not by the public. A riparian landowner may have the privilege of diverting half the flow of a stream, subject to the "reasonable use" or requirements of owners lower down the stream, provided that he returns the escape-water from his canal to the river within the boundaries of his farm. Water may not be passed on to the next farm lower down without the consent of all riparian owners on the stream. This clause appears, however, to have been more honoured in the breach than in the observance. It is thus possible for upper riparian owners to exhaust the supply, to the detriment of those lower down, whilst the latter can obtain redress only through the law-courts. It provides for the registration of water-rights on the title-deeds of property. Great value has naturally been attached to these rights, and they have been bought and sold as such, apart from the value of the land accompanying them. However desirous

the Government may be to utilize the water flowing in streams for the good of the majority and the advancement of agriculture, it cannot fail to recognize rights so acquired. Down to the present time no attempt of any consequence has been made by riparian owners to make use of "flood-waters." The Government therefore can safely declare such waters to be the property of the people, and can store and utilize them for the common good.

Before the Government can adopt an enterprising irrigation policy, however, it is necessary to modify and extend the existing laws to provide for:—

(*a*) Definition and registration of all existing rights once for all.

(*b*) The declaration of all "flood" and unutilized perennial waters as the property of the people.

(*c*) Administrative control of the water in important streams, &c.

(*d*) Provision for the grant of privileges in the future, under Government control.

(*e*) Correct definition of terms used in the law.

(*f*) Expropriation of land necessary for public works.

(*g*) Power to levy and collect water-rates.

(*h*) Jurisdiction of the law.

(*i*) Definition of offences, and the penalties attached.

The first step in the administration of this law must be the correct gauging of the perennial flow of important streams, and the limitation of the existing rights to the capacity of this flow. In this connection an official cadastral survey of the course of the more important perennial streams is desirable. Such a survey would not be expensive, as it would include riparian farms only. It might be plotted to a scale of 8 inches to 1 mile, and would show existing canals or "furrows," and all irrigated lands. The register accompanying it would serve as a record of existing rights, and would greatly simplify future administration. Before the construction of a reservoir by Government is undertaken it is essential that each riparian owner on the stream should have his rights clearly defined, and should understand them. Thereafter it will be possible for the Government to utilize the stored supply to the best advantage, and to obtain a revenue which will justify the expenditure. In order that the working of such a law might be made popular, its jurisdiction might be put into the hands of local Water Courts, whose functions would, however, be purely *judicial* and not administrative. Such courts would settle disputes and try cases of breach of the law brought before them by the department responsible for the administration or executive control of the water-supply. In special cases the local administrative officer or Water Bailiff might be invested with the

powers of a magistrate to take cognizance of offences upon complaint, subject to appeal to the Water Court. It is doubtful whether the people of the country will at the present time permit official control of the water in streams, although such control might be to the advantage of the community. In January, 1905, the Government appointed an influential Commission to inquire into the subject and to recommend the necessary legislation. This Commission has not yet presented its Report.

Climate.—For purposes of irrigation the Transvaal may be divided into three regions, according to altitude, namely:—

(*a*) The High Veld—more than 5,000 feet above mean sea-level.

(*b*) The Mid-Veld—between 4,000 feet and 5,000 feet above mean sea-level.

(*c*) The Low Veld—less than 4,000 feet above mean sea-level.

The High Veld comprises the districts of Bethel, Ermelo, Heidelberg, Standerton, Wakkerstroom, Witwatersrand, and parts of Carolina, Lydenburg, Middleburg, and Pretoria. The annual rainfall in these districts is generally 25 inches to 30 inches, the greater portion of which falls during the months of January, February, and March, *i.e.* in the autumn, which is followed by a frosty winter and a very dry spring. Thus under natural conditions all plant-life is placed at a disadvantage. With the aid of irrigation and ample fertilization of the soil, good crops, such as wheat, oats and lucerne, can be grown, and provision can be made for feeding stock in the winter. These lands, however, owing to their high altitude, do not hold out great prospect of development by irrigation.

The Mid-Veld comprises generally the districts of Bloemhof, Lichtenburg, Lydenburg, Middleburg, Piet Retief, Potchefstroom, Pretoria, Swazieland, and Wolmaransstad. The annual rainfall may be taken as varying between 20 inches and 25 inches. At a lower altitude, and with less severe winters than the High Veld, these lands are capable of great development under irrigation. The climate at the same time is such that Europeans can thrive in it.

The Low Veld comprises generally the districts of Barberton, Marico, Rustenberg, Waterberg, and Zoutpansberg. The annual rainfall, according to recent records, varies in different parts between 20 inches and 60 inches. Severe frosts are not experienced in the low-lying parts, and it will be possible with the aid of irrigation to introduce sub-tropical agriculture, and to obtain two crops in the year. Close acquaintance with the region does not show it to be as unhealthy to Europeans as it unfortunately has the reputation of being. The Drakensberg Range, running north and south, divides the low country and affords on its heights a bracing climate.

Necessity for Reservoirs.—The irrigation engineer of Northern India would class the "spruits" of the Transvaal as "nullas" or drains for passing off flood-water. The pride with which the local inhabitant points out the beauties of a stream carrying a perennial discharge of 40 cubic feet per second indicates the great value attached to water. Such perennial flow as exists is derived solely from springs. Streams of any consequence which lie to the west of the Drakensberg all derive their supply from the dolomite formation. The extent of this formation is very clearly shown in the Geological Map of the Transvaal by Dr. Molengraaff. In the following Table are given the discharges of the principal streams the perennial flow of which has been practically utilized in irrigation by riparian owners:—

Name of Stream.	Approximate Perennial Discharge Cubic Feet per Second.
Mooi River	60
Schoonspruit	40
Klip River	7
Aapies River	9
Maghalies River	16
Sterksstroom	8
Marico	40
Crocodile River	30

Such other streams as have a useful perennial flow lie either to the east of the Drakensberg or remote from the centres of population. Without prejudice to the eventual development of irrigation all over the Transvaal, it appears advisable in the first instance to carry out schemes as near Pretoria and Johannesburg as possible. The perennial flow of streams near the centres of population has, however, been absorbed by riparian owners, and further development must depend on storage-reservoirs. The Vaal and Great Oliphants rivers, moreover, practically run dry in November.

Existing Irrigation-Works.—Previously to the declaration of peace in South Africa, when the Transvaal became finally a British Colony, irrigation had been carried on almost entirely by private individuals. Nearly all riparian farms had their own little canals or "furrows." In some cases two or three farmers had combined to take out a joint furrow, but in every case the method employed was simple diversion of the perennial flow of a stream. All these channels, carrying a discharge of 10 cubic feet per second, or less, would have been classed in Northern India as "minors." A few larger canals have been made since the British occupation. The Government has constructed two small canals, the Potchefstroom Government Canal and the White River Canal, which will be described later.

Future Development.—The progress of the country depends on irrigation. A million pounds sterling could be spent in carrying out profitable schemes already investigated, without exhausting the possibilities of the country. A list of a few of the possible schemes is given in the following Table:—

		1	2	3	4	5	6
	Name of Project.	Estimated Gross Cost of Project.	Amount of Storage.	Storage Rate.	Area Irrigable.	Cost per Acre Irrigable.	Enhancement in Value of Land when Brought Under Irrigation.
		£	Million Cubic Feet.	£ per Million Cub. Ft.	Acres.	£	£ per Acre.
1	White River Canal.	16,617	perennial flow	..	1,700	9·6	10
2	Great Letabo Canal	29,177	ditto	..	3,500	8·30	10
3	Hartingsburg reservoir project . .	51,626	63	708	600	86·00	25
4	Wolmaransstad reservoir project .	34,172	75	427	600	56·95	25
5	Schweizer Reneke reservoir project .	29,152	187	143	660	44·17	20
6	Mooi River reservoir project (near Potchefstroom) .	153,395	743	178	7,500	20·45	15
7	Oliphant's Poort reservoir project (near Rustenburg)	173,240	1,195	118	7,500	23·00	10
8	Christiana reservoir project . . .	419,319	2,187	131	17,000	24·6	10
9	Klip Kopjies reservoir project . .	25,000	100	150	1,000	25	10
10	Schoonspruit reservoir project . .	100,000	500	150	4,000	25	20
11	White River, right bank extension .	5,000	..	..	900	5·5	10
	On the Vaal River—						
12	Klip River, O.R.C., reservoir . . .	283,000	4,165	..	21,600	13·10	10
13	Koppiesfontein reservoir . . .	945,000	5,545	..	60,000	15·70	10
14	Lindiques Falls reservoir . . .	285,000	2,404	..	18,000	15·83	15
15	Coal Mine Drift reservoir . . .	515,000	2,604	..	26,000	19·8	15

Extracted from the Transvaal Administration Report, 1904–05.

The figures given for projects 1 to 8 are based on detailed estimates; the others are approximate. The storage rate (column 3) has been

calculated on the cost of the reservoir and not on the gross cost of the project (column 1). The storage rates appear at first sight to be unusually high, but this is due to the high rates which it is necessary to pay for work done in the country. The cost of storage in the Bombay Presidency varies between £12 and £152 per million cubic feet,[1] being about one-seventh to one-fourth of the cost in the Transvaal.

The cost per acre irrigated (column 5) must be considered in conjunction with the enhancement in value of land when brought under irrigation (column 6). It is an interesting fact that in some cases the capital cost is covered by this enhancement. At the present time irrigated land is in many places valued at £40 to £100 per acre, but this can only be accepted as an inflated value due to the small area of the land, and to the fact that farmers do not wish to dispose of it.

From an engineering point of view there should be no serious difficulty in carrying out all or any of the projects mentioned.

Nos. 1 and 2 are simple masonry weirs diverting a perennial supply.

Nos. 3, 4, 5, 6, 9, 10, and 12 are storage-reservoirs with earthen embankments.

Nos. 7 and 13 are masonry dams 115 feet and 69 feet in height respectively. Nos. 8, 14 and 15 are storage-weirs on the Vaal River.

The list does not exhaust the schemes which are possible; rivers like the Great Oliphants, Marico, and Crocodile have not yet been investigated.

Replenishment of Reservoirs.—Owing to the fact that no reliable hydrographic records have been kept, the probable yield from a catchment can be arrived at only by estimation and by comparison with other countries. The yield of the run-off from catchment-areas will differ very much from that in a country like India, for example, where it varies between 10 per cent. and 30 per cent. of the total rainfall.

The chief factor which influences run-off is evaporation. This is abnormally high in the Transvaal, where the effects of a shower of rain are obliterated by sun and wind in a very short space of time. Another factor is the short duration of the falls. The rainfall is distributed over 9 months in the year, during 4 months of which the fall is usually less than 1 inch. The following Table has been compiled from an actual record of a year's rainfall, taken at Government Buildings, Pretoria, which is fairly typical of the rainfall of the Transvaal:—

[1] Return published by the Bombay Government.

Month.	Rainfall.	Number of Days on which Rain fell.	Average Fall per Day on which Rain fell.
1903	Inches.		Inches.
July	0·00	0	0·0
August	0·00	0	0·0
September	0·21	2	0·1
October	1·25	5	0·25
November	5·53	13	0·43
December	4·07	12	0·34
1904			
January	3·96	10	0·4
February	5·45	13	0·42
March	7·40	14	0·53
April	0·38	5	0·07
May	0·10	1	0·10
June	0·07	1	0·07
	28·42	76	0·37

Assuming that the rainfall is evenly distributed during the period of its fall, which is not the case, the greatest fall during 24 hours is 0·53 inch, which would produce practically no run-off at all. In the foregoing record, falls of more than 1 inch occur on 9 days in the year. Assuming that falls of less than 1 inch would produce no run-off, an estimate of the probable yield is given in the following Table:—

Date.	Rainfall.	Probable Percentage of Run-off.	Yield.
	Inches.	Per Cent.	Inches.
4 November	1·04	20	0·208
20 "	1·04	"	0·208
22 "	1·16	"	0·232
1 December	1·32	"	0·264
20 January	1·35	"	0·270
25 February	1·91	"	0·382
3 March	2·53	50	1·265
9 "	1·07	20	0·214
12 "	1·41	"	0·282
	12·83	25·9	3·325

The estimated yield, namely 3·325 inches, is equivalent to 11·7 per cent. of the total rainfall of 28·42 inches. This estimate, it is thought, errs on the side of liberality, and in general, in the preparation of projects, a greater yield than 7 per cent. should not be relied upon. It will be some years before the conditions pertaining to particular catchments, and the influence of the geological formation on the yield, can be accurately determined.

Discharge due to Run-off.—The highest rainfall recorded during a short period was that on 18 November 1904, when 3 inches of rain fell in one hour at Belgravia, Johannesburg. During the Bloemfontein Flood disaster, which was witnessed by the Author, rain fell for 2½ hours at a rate probably exceeding 2 inches per hour. As the ground had previously been saturated by rain, a very large percentage of the fall ran off the ground, and the run-off was augmented at the time of maximum flood by the overflowing and bursting of a dam just above the town. This dam, when the failure took place, contained about 10 million cubic feet of water. It was situated at a point where the drainage from four parts of a catchment-area of 13·83 square miles converged. The volume of the flood was calculated to be 22,222 cubic feet per second, of which 15,570 cubic feet per second is attributed to a run-off of 1·75 inch, and 6,652 cubic feet per second to the bursting of the dam.[1]

The rainfall and the discharge due to the run-off were abnormal for the locality, although only about equal to what would be allowed for in India, where for a similar catchment a discharge of 15,457 cubic feet per second would be provided for, which is equivalent to an average run-off of 1·69 inch from the catchment.[2] In South Africa, however, a flood of such intensity is a rare occurrence. Heavy falls are generally confined to small areas, and are of short duration, and these two factors must be taken into consideration if works are to be designed which may be executed at a reasonable cost. In the case of very small works, having a catchment-area of less than 15 square miles, the cost of repairing the damage done by a breach once in 20 years would probably be less than the interest on the capital required to provide for such floods, while in the case of large works the capacity of the reservoir would probably be sufficient to afford a safeguard, owing to the short duration of the floods. In localities, however, where possible failure may have disastrous results in loss of life and property, in addition to the loss of the crops depending on the reservoir, it will always be better to err on the side of safety by

[1] Official Report by Mr. Gerrand.

[2] W. L. Strange, "Indian Storage Reservoirs with Earthen Dams." Appendix 8. London, 1904.

providing for the maximum flood. In the present state of knowledge of the country any estimate of the intensity of the run-off must be approximate only. The country has now been divided into hydrographic districts, and observations of rainfall and the discharge of streams are being carefully recorded by the Government.

Until fuller hydrographic information is available the discharges given in the following Table[1] may reasonably be allowed; they must, however, be regarded as tentative only.

Increments of Catchment Area.	Run-off from each Increment of Catchment Area.	Discharge from each Increment of Catchment Area due to Run-off.	Discharge from Total Catchment Area due to Run-off.	Average Run-off from Total Catchment Area.
Square Miles.	Inches per Hour.	Cub. Ft. per Second.	Cub. Ft. per Second.	Inches per Hour.
0–1	1·50	968	968	1·50
1–2	1·20	774	1,742	1·35
2–3	1·00	645	2,387	1·23
3–4	0·85	548	2,935	1·14
4–5	0·70	452	3,387	1·05
5–6	0·60	387	3,774	0·97
6–7	0·50	323	4,097	0·91
7–8	0·45	290	4,387	0·85
8–9	0·40	258	4,645	0·80
9–10	0·35	226	4,871	0·76
10–15	0·25	806	5,677	0·59
15–20	0·20	645	6,322	0·49
20–25	0·16	516	6,838	0·42
25–50	0·12	1,936	8,774	0·27
50–75	0·10	1,613	10,387	0·22
75–100	0·09	1,451	11,838	0·19
100–150	0·07	2,258	14,096	0·15
150–200	0·06	1,936	16,032	0·12

Where a catchment-area is of dolomite formation the run-off would be much less than that given by the Table.

The Duty of Water.—The duty of water has a very direct bearing on the cost of irrigation projects; it is therefore of great importance to determine what would be a fair duty to apply to the country. It is proposed to take a plot of 4 acres and farm it with suitable crops at suitable seasons, and to deduce therefrom the duty of the water used. The plot may be divided into four parts of 1

[1] Transvaal Department of Irrigation, Bulletin No. 2.

acre each. The following Table shows the months during which the various crops would be in the ground.

Month.	Plot I.	Plot II.	Plot III.	Plot IV.
January	Potatoes	Tobacco	Vegetables	Lucerne
February	,,	,,	,,	,,
March	,,	,,	,,	,,
April	Wheat or Oats	Barley or Rye	,,	,,
May	,,	,,	,,	,,
June	,,	,,	,,	,,
July	,,	,,	,,	,,
August	,,	,,	Potatoes	,,
September	,,	,,	,,	,,
October	,,	Tobacco	,,	,,
November	,,	,,	,,	,,
December	Potatoes	,,	,,	,,

This is to be regarded as typical of one season; the cropping during the following season would have to be varied. Plots 1, 2 and 3 are shown as producing two crops a year, but in practice a small area would lie idle for a few months, so that a fair estimate would be five instead of six crops in one year from the three plots. Lucerne would remain permanent for a number of years; the ground would then be broken up, and probably would be sown with potatoes, oats, or wheat. Meanwhile another plot would be sown with lucerne. In a dry season the number of waterings to a depth of 4 inches necessary to bring the crops to maturity would be as follows:—

Plot.	Crop.	Number of 4-inch Waterings.
1	Wheat or Oats	6
	Potatoes	3
2	Barley or Rye	6
	Tobacco	4
3	Vegetables	4
4	Lucerne	7
	Total	30

The second crop (potatoes) in Plot 3 has been omitted from the calculation for the reason already explained. Based on the foregoing

consumption the duty of the water flowing on to the crop would be 289 acres per cubic foot per second. Allowing for 50 per cent. loss in transit the duty at the head of a canal would be 145 acres per cubic foot per second. On a larger area the cropping would be on a less intensive plan, and consequently less water would be used and the duty would be larger. In the preparation of projects, however, it would be advisable to take a duty of 150 acres per cubic foot per second as applicable to the country, which is equivalent to a storage of 210,240 cubic feet for each acre irrigated, or nearly 5 acres to every million cubic feet stored.

Water-Rate.—At existing market rates, and assuming a fair crop to be obtained with the use of manures and fertilizers, the value of the crops grown on the four plots of 1 acre each would be as follows:—

No.	Value of Crops, per Acre.	£	Total Cost of Production, per Acre.	£
1	Wheat or Oats	10	Cost of Seeds and Manures	40
	Potatoes	20	Land Rent	12
2	Barley or Rye	10		
	Tobacco	20		
3	Vegetables	15		
4	Lucerne	20		
	Total	95	Total	52
			Profit	43

It is generally reckoned that one crop in five is destroyed by locusts, hailstorms, etc. The occupier's net profit will therefore be £34 or £35 on the four plots, or £8 or £9 per acre. Assessing the water-rate at 10 per cent. of the net profit, which is the ratio obtaining in India, the occupier of irrigated land should be able to pay 15*s*. to £1 per acre per annum, in addition to paying a land-rent of £3 per acre. This land-rent is calculated as 10 per cent. interest on £30, the value of an acre of land under water. A liberal estimate of the value of good unirrigated land would be £3 to £5 per acre. Each acre of land is enhanced in value by £25 by being brought under irrigation. It would therefore be a safe investment for Government to buy the land which would be benefited by an irrigation scheme, and let it out in suitable plots at a combined water- and land-rent sufficient to cover the interest on capital outlay, with a margin of profit for maintenance. The tenant would

benefit, as he would not be charged interest on the full enhanced value of the land. In the event of Government being unable to purchase the land, it would be fair to charge the landlord an additional "owner's rate" based on the enhanced value of his land.

Bearing of Irrigation on Settlement and Population.—Inquiry is often made as to the number of families which any given irrigation-scheme will support. Politically and socially it has a very direct bearing on the future of the country, which will depend on whether the poor white man can settle in it and successfully farm a small area of land. The cost debars him from taking up large areas and stock-farming as the Boer farmer does. An income of £200 per annum derived from his land only, with the addition of poultry and stock, should be sufficient to maintain a man and his family. Therefore, with a profit of £9 per acre the irrigated area required to support one family should be 22 or 25 acres: *i.e.*, the duty of 1 cubic foot per second would be the support of six families; or a storage of 5 million cubic feet of water would support one family.

Important as it is to put the land under irrigation, it is equally necessary to find the best class of settler to work the land afterwards, and to complete what the engineer has only begun. In India and Egypt there exists a teeming population with established methods of agriculture handed down from generation to generation. The engineer there is not called upon to instruct the agriculturist as to how he should manipulate the supply given to him. The white man expects the water to be led for him to each plot of his land; and he will recognize nothing but "flow" irrigation. Although he may work hard himself, most of his ploughing and similar work will be done by hired Kafir labour. It would be a moderate estimate to allow one native labourer for every five acres irrigated. Thus 100,000 acres put under irrigation would necessitate the employment of 20,000 natives, and this in the present state of the labour market would be difficult to accomplish. A policy of gradual irrigation development is therefore advisable.

Imports.—A consideration of the value of agricultural produce imported into the country will indicate the leeway which the country has to make up in order to be self-supporting. Purely agricultural products, which could be produced under irrigation in the country, were imported during each of the years 1903 and 1904 to the value of 2½ million pounds sterling. In addition to this, such articles as butter, cheese, eggs, cigars, etc., which indirectly bear on agriculture, are imported annually to the value of half-a-million sterling. Although the country may never be capable of exporting agricultural produce, it is quite capable of producing sufficient for its own consumption.

Potchefstroom Government Canal.

This work was begun under instructions from the Land Department to provide work for indigent burghers, and to irrigate the lands of the Government experimental farm and settlement at Potchefstroom. It was subsequently completed under the supervision of the Department of Irrigation and Water Supply. It takes off as a branch from the existing Town Canal with a head-discharge of 20 cubic feet per second, and is 9½ miles in length. It was intended to irrigate an area of 2,600 acres, but owing to disputes with the Town Council as to the quantity of water to be diverted it has not yet been worked up to its full capacity. In order to gain as much command as possible, it was given a very low longitudinal land-slope, viz. 0·00017 for the first 4 miles from the head, and 0·0001 thence to the tail; this slope has been found to be too low. It is a single-bank canal, and a considerable amount of silt is brought into it by cross-drainage, which, owing to the low velocity in the canal, is quickly deposited in the bed, making the annual maintenance very high. It would have been better to have made the longitudinal slope 1 foot per mile, increasing to 2 feet per mile, and to have extended the area under irrigation towards the Mooi River.

It is doubtful whether a single-bank canal is in the end more economical than one having two banks. In Africa the management of the cross-drainage is the chief difficulty with which the engineer has to contend. It is seldom or never possible to align the canal to run on a watershed, although this can usually be done with the distributaries or "water-courses." The cross-slope of the ground surface varies, in general, between 1 in 6 and 1 in 20. The initial expense of a single-bank canal is comparatively small, as the excavated earth forms of itself a bank on the lower side. A double-bank canal not only involves a low bank on the upper side, but necessitates catchwater-drains to lead the drainage to points where it can cross the canal. The ratio between the costs of a single-bank and a double-bank canal is approximately 1 to 1·5, but it would vary with the cross-slope. Economy in maintenance of the latter would in a few years make the costs practically equal. A single-bank canal should not be made in the neighbourhood of a town where there is much traffic to cut up the ground surface, which is carried into the canal by the wind and rain, but it may be justifiable in a district remote from settlement and covered with vegetation.

In laying out the land to be irrigated, it would be better to measure the area from the point where flow-irrigation becomes

possible, instead of from the canal-bank, where the ground surface for a short distance is necessarily above full-supply level. "Lift-irrigation" as practised in India and Egypt is as yet undeveloped in South Africa.

White River Canal.

This canal has been constructed recently by the Government, and as it is a typical example of the class of work which engineers in the country may be called upon to carry out, it will be described in detail.

Source of Supply.—The source of supply is the White River, a perennial stream which takes its rise in the hills about 40 miles north of Barberton. The canal takes off from it at a point about 13 miles from its source. The perennial discharge of the stream varies between 12 and 18 cubic feet per second. The catchment-area above the intake is about 40 square miles, and the rainfall on the basin is about 35 to 40 inches per annum.

Area under Command.—The area under command consists of a series of undulating ridges with depressions between, necessitating in the alignment of the canal some form of cross-drainage at each depression. These ridges have a general cross-slope of 1 in 10. The area commanded by the canal in the first $10\frac{1}{8}$ miles is 1,718 acres. By extending the canal another 4 miles an additional 800 acres can be brought under water, and this is now being done, Fig. 1, Plate 7.

Alignment.—The alignment of the main canal necessarily follows a falling contour along the sloping country, and never attains the watershed.

Cross-Section and Longitudinal Slope.—It was intended ultimately to construct a tail-reservoir at 10 miles 6 furlongs (Fig. 1, Plate 7), so that the night-flow of the canal could be stored for use lower down; for this reason the cross-section of the canal (Fig. 2, Plate 7) was made uniform throughout this length. The longitudinal bed-slope of 1 in 2,000 was also kept unchanged. The full-supply depth was 2 feet and the discharge 20 cubic feet per second.

Headworks.—The headworks consist of a low diverting masonry-weir, 125 feet in length, with a head-regulator at right-angles to it, to control the supply entering the canal. A length of 50 feet of the weir on the left bank was built with its crest 2 feet lower than that of the remainder of the weir, and an angle-bar sill was fixed to the top in order that it might act as a clear over-fall gauging-weir. This portion of the weir was separated from the rest by a pier, Figs. 3 and

4, Plate 7. The object of the pier was to obviate end-contraction in the water flowing over the gauging-weir and to admit air below the falling water down-stream of the crest. The remaining length of 75 feet of the crest was built in masonry, 2 feet in width at the top, and would come into action as a drowned weir when the floods exceed the discharging-capacity of the lower crest. The extreme left flank was also raised 2 feet higher than the lower crest and was carried 5 feet into high ground. On the right flank the weir was connected with the three scouring-sluices, beyond which, and at right-angles to them, was built the head-regulator, which was provided with two openings, 4 feet in width by 2 feet 6 inches in height, having in them wooden sluice-gates, worked by lifting-screws and capstan-heads, for regulating the supply in the canal. The whole work is securely founded on rock, and ample provision is made for any floods likely to pass over it.

Cross-drainage Works.—The cross-drainage works are of two kinds, namely, trough-aqueducts and canal-dams.

Trough-aqueducts were provided where the drainage passing the canal was more than could be economically dealt with in any other way, and where the contour of the ground did not lend itself to the construction of canal-dams. Trough-aqueduct No. 1 is 280 feet in length, whilst Nos. 2, 3 and 4 are each 60 feet in length between abutments. They are supported on trestles spaced 20 feet apart, centre to centre, Figs. 5, Plate 7. No. 1 is carried partly on iron trestles and partly on stone piers, the latter being used where the foundations were considered too insecure for trestles. The use of piers and trestles together has somewhat detracted from the appearance of the work, but that is a matter of small importance in this out-of-the-way region. It will be advisable in future works of this nature to use stone cut-water piers to a certain height, and to make up the total height with trestles, firmly fixed to the tops of the piers. This arrangement would present a more uniform and pleasing appearance. The trough was found to expand $\frac{3}{8}$ inch at each end, or a total expansion of $\frac{3}{4}$ inch in its length of 283 feet. The water-tight joint at the ends of the trough, on the abutments, is shown in Fig. 6, Plate 7. This joint was not altogether satisfactory, owing to the extremes of temperature making it difficult to keep the composition of pitch, tar, and sand sufficiently viscous. In aqueduct No. 4 the expansion-joint was altered to that shown in Fig. 7, Plate 7, which has been found quite satisfactory.

The cost of the ironwork, as finally erected, and inclusive of al charges, amounted to £2 19*s.* per foot run of aqueduct, or 6*d.* per pound. It was thought advisable to have it manufactured locally

in order to encourage local enterprise, and to have the work carried out quickly.

Canal-dams were provided in five places where the catchment-area of the cross-drainage was less than 1 square mile. They consist of simple embankments across the "spruits" with sufficient waste-weir provision on one flank to pass off floods. The level of the crest of the waste-weir was in every case fixed at the full-supply level of the canal. In cross-section the embankment is 6 feet in width at the top and 4 feet above highest flood-level, or 5 feet above the full-supply level of the canal, with a 2-to-1 upstream or water slope and a 1½-to-1 downstream slope. The slopes are reduced to 1½-to-1 and 1-to-1 respectively where the height of the embankment is less than 8 feet. A puddle-trench is constructed along the centre-line of the embankment.

Escapes.—Four escapes are provided at convenient intervals along the canal for safety during rainfall, when surface-drainage water flows into the canal from above. Each of these consists of a simple masonry wall with a rectangular notch in it, the flanks being raised higher and carried into the bank on each side. The notch is provided with double grooves for planks.

Miscellaneous Works.—Level crossings were provided instead of bridges, owing to the greater expense involved by the latter and because the traffic was expected to be small, whilst the depth of water was only 2 feet. The approaches to the canal-bed were given a gradient of 1 in 10, and the bed and slopes were pitched. The width of the drifts was 10 feet.

Catchwater-drains were necessary in order to intercept the surface-drainage on the upper side of the canal and carry it to points where it could enter the canal by inlets, or be connected with the cross-drainage. The cross-section of the drains varied between 1 foot by 9 inches and 4 feet by 2 feet, and the longitudinal slope was maintained as far as possible at 1 in 1,000. Pitched inlets into the canal were provided where necessary.

Distributaries.—Owing to the ground-surface sloping away from the canal at a gradient of about 1 in 10, the alignment of the distributaries could not be carried along the minor watersheds, as the velocity produced by the excessive slope would have scoured out the soil; and to have reduced this slope by falls and rapids would have involved too heavy an expenditure. The alternative was therefore proposed of running the distributaries parallel with the canal, leaving the irrigator to arrange his own distribution from any point along the distributary. It was decided, however, to postpone the excavation of the distributaries, with the exception of that leading through the

township, and to provide a temporary pipe-outlet for each settlement-plot. The settler using the water would thus be free to make his own arrangements for distribution, and, if found necessary later, the site of the outlet could be moved without difficulty or expense. The distributary leading through the township was carried out and the excessive slope was overcome by providing occasional pitched rapids.

Rates.—The rates at which the work was carried out, and also, for purposes of comparison, those which would be paid for similar work in India and Egypt, are given in the following Table:—

Description.	Unit.	Rate at which the Work was carried out.	Approximate Rate which would be Paid for Similar Work in India in English Currency.	Approximate Rate which would be Paid for Similar Work in Egypt.
		£ s. d.	£ s. d.	£ s. d.
Excavation in soil . . .	100 cubic feet	0 4 7½	0 0 6 0 0 8	0 1 3
Excavation in rock . . .	100 cubic feet	0 16 0	0 7 8	0 11 0
Pitching, 12 inches . . .	100 square feet	3 15 0	0 8 0	1 14 0
Masonry-coursed rubble .	100 cubic feet	12 8 10½	2 2 8	2 16 0
Concrete	100 cubic feet	(not used) Estimated rate £10	1 6 0 (lime) 4 13 0 (cement)	2 11 0

The Paper is accompanied by three drawings, from which Plate 7 has been prepared.

(*Paper No. 3601.*)

"Counterweight Device on the Balmain Tramway, Sydney, New South Wales."

By Percy William Shaw, M. Inst. C.E.

In 1902, the Government of New South Wales authorized the extension of the Balmain Tramway, Sydney, from Gladstone Park to the Darling Street wharf, $\frac{3}{4}$ mile distant, from which a line of ferry-boats maintains a regular service with the City. The extension, which is operated electrically, was constructed and opened for traffic as far as Nicholson Street in November of the same year. From this point the ground falls rapidly to the wharf, a distance of 10 chains, the first 3 chains of which are on a gradient of 1 in 10·54, whilst the remainder is on a decline of 1 in 8·24. Although in a few instances gradients as steep as 1 in 8 have been successfully introduced in electric lines depending on adhesion only, it was not deemed advisable, under the circumstances, to adopt this course. From experience gained on other lines of the system, with gradients as steep as 1 in 10 running up from the water's edge, the Railway Commissioners, who control the traffic on tramways as well as on railways in New South Wales, decided that it was not advisable to construct tramways on gradients steeper than 1 in 12, and this decision has been adhered to as closely as possible ever since by the Construction Branch. In order to obtain this gradient from Nicholson Street to the wharf several circuitous routes were considered, but owing to the excessive cost of the land required to carry them out they were finally abandoned, and it was decided to continue the line directly to the wharf on the steep gradient, and to assist the ascent and control the descent of the cars by means of a counterweight. The Author received instructions to prepare a scheme on this principle in November 1902. His scheme was approved, and authority for the construction of the works was obtained in February 1903; the line was completed and opened for traffic in the following November.

The main object aimed at in the design of the counterweight device was to secure safety and ease in operation, whilst at the same time permitting any car in service on the Sydney Tramways to be used on the Darling Street section without any extra attachment or alteration. The counterweight was considered simply as a means of reducing the steep gradient to one which could be safely operated with any car and by any driver, so that it should not be necessary to employ only picked men. The results have proved entirely satisfactory, the line having now been in constant use, on a 20-minute service, for more than two years, during which time it has received some fairly rough handling from inexperienced and nervous drivers.

The general arrangement of the running-track and counterweight-subway is illustrated in Figs. 1, Plate 8. The descending car, on coming on to the incline, is brought into contact with a buffer-trolley, which it carries along in front of it. The buffer-trolley is attached by a steel-wire cable, passed round a sheave at the top of the incline, to a counterweight-trolley running on rails in a subway under the incline, so that as the car descends the counterweight-trolley ascends, acting as a brake on the descending car. At the foot of the incline a buffer-stop prevents the car over-reaching and carrying the counterweight too far up. On the return-trip the car is started in the ordinary way by means of the electric motors, and the trolley, now at the rear end of the car, exerts a constant force upon it due to the pull of the descending counterweight, and thus assists the car up the hill. On arriving at the top the trolley comes to a standstill by the counterweight arriving at the foot of the subway, where it is brought to rest by a hydraulic buffer in which the momentum of the moving weight is absorbed.

Permanent-Way.—As the roadway is only 32 feet in width between the curbs, the centre-line of the surface-track was laid 2 feet 6 inches out of line with the existing track, so as to bring it 9 feet from the curb and leave as much room as possible free for vehicular traffic. The track consists of grooved rails weighing 83 lbs. to the yard, in 30-foot lengths, resting on a bed of concrete 6 inches in thickness, Figs. 2, Plate 8. The rails are anchored down by means of two short lengths of T-rails, weighing 60 lbs. to the yard, bolted on to the bottom flanges 6 feet from the end of each rail. The gauge of 4 feet 8½ inches is maintained by tie-rods bolted through the webs of the rails at every tube-frame, and the rails are bonded with copper bonds at every joint. The cast-iron tube-frames, spaced 3 feet 9 inches apart, are bedded in concrete, and support the slot-beams, which are bolted on with two ¾-inch bolts on each side. The

slot-beams are in 15-foot lengths, laid so as to break joint, and secured by two bolts at each end. Carrier-sheaves, 12 inches in diameter, fixed at intervals of 30 feet in pockets below the level of the cable-tube, support the cable at a height of 6 inches above the bottom of the tube.

Subway.—In most inclines operated by means of a counterweight running in a subway, the subway has been constructed under the running-track. In some cases the subway follows the contour of the line, and the rails are laid directly on the roof, without packing; but this method has not proved satisfactory, and much better results have been obtained where the subway has been laid on an even gradient, and at a sufficient depth to permit of the sleepers being ballasted and packed in the usual manner. Owing to the rocky nature of the ground through which the subway would have to be cut, and the small depth available for drainage at the foot of the incline, it was considered advisable to place the subway on one side, at a distance of 6 feet from the surface-track, centre to centre, Figs. 2, Plate 8. The excavation showed very broken ground; in places it had been made up to about 5 feet in depth, whilst the greater portion consisted of sandstone rock, which cropped up to within a few inches of the surface. This rock was of a very hard nature, especially where it had been impregnated with iron. As the use of explosives was not allowed, the excavation had to be taken out by picking and gadding, and this proved very expensive work. The counterweight-trolley track is of 2 feet 6 inches gauge, and is laid on an even gradient of 1 in 8·48, at a minimum depth of 4 feet 6 inches below the level of the surface-track. The rails, of T-section weighing 60 lbs. per yard, are laid in 30-foot lengths, spiked to longitudinal sleepers of ironbark timber, 10 inches by 5 inches by 15 feet in length. The sleepers are bedded in concrete and held down by means of iron dowels, 1 inch in diameter by 16 inches in length, fixed 3 feet from each end. Carrier-sheaves, similar to those in the surface-track, are set in the centre of the track, spaced 30 feet apart.

Sheave-Pit.—At the top of the incline the cable connecting the buffer-trolley with the counterweight passes round a sheave, 6 feet in diameter, set in a pit 12 feet 4 inches by 9 feet 1 inch and 5 feet 3 inches in depth below the surface of the road, Figs. 3, Plate 8. A sheave-trap is provided, composed of castings bolted together in convenient lengths for handling, and filled in with tarred metal. The sheave is built in halves, and should an accident occur, necessitating its removal, it can be taken through the trap, which can be readily opened for that purpose.

Before the line was handed over to the Railway Commissioners the Author had a car at his disposal for the purpose of making a series of trials. On one occasion he had the car taken over the hill at full speed. The buffer-trolley on this occasion ran right up to the end of the slot-way, where it was only stopped by the grip-shank coming in contact with the end of the slot-way, causing about 12 feet of slack to collect on that side of the sheave, but the cable was effectually prevented by the guard from coming off the sheave. To prevent the trolley from travelling too far in the future, a spring-stop was fixed, about 3 feet from the end of the slot-way, under the slot-beams, and bolted to them. The stop is made and fixed so that its rod meets the grip-shank as close to the slot-beams as possible. Cast-iron training-sheaves, 15 inches in diameter, are set on each side to lead the cable on to the 6-foot sheave; these are also provided with a hoop-guard, to prevent the cable from being jerked off in the event of any slack being formed by unevenness in the tension of the cable. The 6-foot sheave is fed with oil through a small brass tube from a lubricator fixed above it, encased in a cast-iron box attached to the upper side of the roof-plates, and having its cover flush with the roadway. This cover is provided with a locking-arrangement so adjusted that the key cannot be withdrawn in any other but the locked position, in order to preserve it from being tampered with by unauthorized persons.

Terminal Pit.—At the end of the incline and close to the wharf a terminal pit is constructed for the accommodation of the hydraulic buffer and counterweight-trolley, Figs. 4, Plate 8. The pit, 23 feet 6 inches in length, is provided throughout its entire length with a removable roof-covering, composed of reinforced-concrete plates, supported on rolled-joist bearers and columns, set in line with the subway. Passages 2 feet in width run along each side of the pit, covered with a fixed roof composed of $\frac{3}{8}$-inch dish-plates, attached to the rolled joists and angle-bars, and filled with concrete. On one side this roof-covering supports one rail of the surface-track, which terminates near its centre in a fixed buffer-stop. After the roof dish-plates had been bolted in position, and the buffer-stop had been erected and fixed, a concrete buttress 3 feet in length was built up from the floor to the underside of the roof, to transmit the thrust on the buffer-stop directly to the solid ground, and thus prevent straining of the roof. The buttress projects 15 inches from the wall, which was picked and roughened to ensure adhesion of the buttress to it. For a distance of 7 feet the side passage under the surface-track is widened to the frames of the cable-tube, and the wall of the tube is cut away, leaving an opening through which the grip on the buffer-

trolley can be inspected whilst waiting at the foot of the incline, before starting on its return journey. The opening is provided with light timber shutters to prevent dust and dirt being blown in from the tube to the hydraulic buffer. As a precaution against spreading, the centre rolled-joist bearers are tied together with 1-inch movable round tie-bars, spaced 5 feet apart, hinged to angle-bar brackets on the joists, and provided with union-screws at the centre for adjustment.

Lighting.—The whole of the subway, including the sheave-pit and the terminal pit, is lighted by electricity. Current is taken from the trolley-wire to a main lighting-switch fixed on a span-pole, from which connection is made with switches fixed at each man-hole, each controlling five 16-candle-power lamps connected in series. The lights in the subway are situated practically at each carrier-sheave, or about 30 feet apart. Each pit has the five lights from its own switch distributed within it, and the whole system is divided into six sections, comprising thirty lamps.

Buffer-trolley.—Running on the surface-track is a small buffer-trolley connected by a steel cable to the counterweight, Figs. 5, Plate 8. The body of the trolley is constructed of ironbark timber, framed, and stiffened with angle-bars. The wheels and axle-boxes are of a standard pattern used on small trailers, and were supplied by the Railway Commissioners. The steel cable running in the cable-tube is permanently attached to a grip by means of a $1\frac{3}{8}$-inch connecting-pin and shackle, with a $1\frac{1}{4}$-inch square shank bolted through the grip-plate. The connecting-pin engages in a thimble securely held by a long splice at the end of the cable. The grip-shank is a $\frac{5}{8}$-inch steel plate, 12 inches in width at the bottom end, where it is wrapped round the square shank of the shackle, and held by three $\frac{3}{4}$-inch bolts. The top end, 18 inches in width, is firmly clipped, by six $\frac{3}{4}$-inch bolts, to a $2\frac{1}{4}$-inch shaft working in cast-iron bracket-bearings bolted to the underside of the framing. The grip-plate is held against the lower end bearing by a helical spring on the shaft, interposed between the plate and the upper bearing. The spring, which is made of $\frac{7}{8}$-inch square steel, and has a mean diameter of $3\frac{1}{2}$ inches and a length of $8\frac{3}{8}$ inches, takes up any undue strain that may be put on the cable. The grip-plate is also free to swing sideways, and to adjust itself in the slot-way. The upper or after end of the trolley has an ironbark timber buffer, 1 foot 9 inches in length, attached to two buffer-rods and volute springs working in cast-iron boxes, spaced 6 inches on each side of the centre-line. The front face of the buffer-beam is hollowed out in the centre, to ensure the curved buffer-plate on the cars striking the beam on each side of the

centre, and as nearly as possible in line with the buffer-rods. The beam is given ample depth to allow for the variation in height of the car-buffers, amounting to about 3 inches. The lower or front end of the trolley has two ordinary side-buffers, with volute springs, spaced 4 feet 9 inches apart. The weight of the cable as it sags between the carrier-sheaves in the subway lifts the trolley about 5 feet farther up the incline after the counterweight has been brought to rest, and the process of gathering up this slack is so gradual that the trolley sustains very little shock from its sudden contact with the descending car, and is carried smoothly forward without apparently offering any resistance. As a precaution against accidents, the trolley is surrounded by a guard suspended from the body and reaching to within 3½ inches of rail-level. The front end is formed into a cow-catcher, whilst the rear end is hung on hinges, forming a gate and giving access to the interior.

Counterweight-trolley.—As already mentioned, the object of the counterweight was to reduce the steep incline to the equivalent of a safe and workable gradient. With this end in view, the weight to be attached was not required to balance the cars, but merely to offer sufficient resistance to enable them to be safely controlled by their own brakes in descending the incline, and to offer some assistance in ascending. The cars running on this line are mostly of the bogie-combination type, 37 feet 6 inches in length over the headstocks, with a 12-foot box-compartment in the centre, and open ends, each containing two longitudinal seats and one cross-seat. They have seating-accommodation for forty-two persons, and weigh, on an average, 15 tons loaded. The counterweight-trolley complete weighs 11 tons, and runs on a gradient of 1 in 8·48, or 11·79 per cent. Taking 22·4 lbs. per ton per 1 per cent. of gradient for the gradient resistance, and 10 lbs. per ton for rolling-friction, the total resistance of the weighted trolley is $\frac{(11 \cdot 79 \times 22 \cdot 4) + 10}{2{,}240} \times 11 = 1 \cdot 34$ ton. This represents the tension on the cable due to the trolley, but its effectiveness is diminished by the resistance due to the weight of the buffer-trolley running on a gradient of 1 in 8·24, or 12·13 per cent. The weight of the buffer-trolley is 1·35 ton, and with the same allowances for gradient-resistance and rolling-friction the total resistance in this case is 0·17 ton, which, deducted from the counterweight's resistance, gives an effective resistance of 1·17 ton. The total resistance of a car weighing 15 tons on a 12·13 per cent. gradient, using similar data, amounts to 1·88 ton. In descending the incline, the friction of the rope and sheaves acts against the car, and assists the action of the counterweight; but in ascending, the

counterweight has to overcome this friction, which acts against it. Allowing 10 per cent. for rope-friction, the effective resistance offered by the counterweight to the descending car would be $1 \cdot 17 + 0 \cdot 11 = 1 \cdot 28$ ton, and to the ascending car, $1 \cdot 17 - 0 \cdot 11 = 1 \cdot 06$ ton. Taking the differences between these resistances and that due to the car, viz., $1 \cdot 88 - 1 \cdot 28 = 0 \cdot 6$ ton, and $1 \cdot 88 - 1 \cdot 06 = 0 \cdot 82$ ton, which are the resistances to be overcome by the car in descending and ascending respectively, and converting these into their equivalent gradients, the following are obtained:—

Effective descending gradient—

$$= \frac{\dfrac{0 \cdot 6 \times 2{,}240}{15} - 10}{22 \cdot 4} = 3 \cdot 54 \text{ per cent., or 1 in } 28 \cdot 25.$$

Effective ascending gradient—

$$= \frac{\dfrac{0 \cdot 82 \times 2{,}240}{15} - 10}{22 \cdot 4} = 5 \cdot 02 \text{ per cent., or 1 in } 19 \cdot 92.$$

It will be readily seen that these effective gradients are only approximate, owing to the varying weights of the cars and the varying effect of the cable according to the relative position of the trolleys.

The underframe of the counterweight-trolley is built of channel-bars, Figs. 6, Plate 8. Two intermediate channels, fixed 2 feet 9½ inches from the headstocks, divide the body into three compartments. The two end compartments are strutted with wrought-iron stays, meeting near the centre of the intermediate channels. A 1½-inch shackle-bolt is provided at one end, for the attachment of the cable, working in a 1¾-inch wrought-iron pipe with screwed flanges bolted to the inside of the headstock, and to the intermediate channel. The shackle-bolt projects into the centre compartment, and is there secured by a nut and washer, screwed against a helical spring. The other end of the underframe is fitted with an ironbark timber buffer-beam bolted to the headstock. The face of the beam is cut to a curve, so that it has a thickness of 6 inches at the centre and 3 inches at the ends; a copper plate is bolted on at the centre and dished to suit the rounded end of the hydraulic buffer. The two end compartments are covered on the underside with ½-inch wrought-iron plates riveted to the frame, and are filled up with lead, run in hot, to within ½ inch of the top of the framing. The whole of the underframe is covered with a 3-inch cast-iron plate, made in

three pieces, projecting 3 inches over the sides. Each section is bolted to angle-bar brackets riveted to the framing. On the underside the plates are cast $\frac{1}{2}$ inch thicker in the centre, forming a projection to take the shear, and made to fit tightly into the inside of the framing. Projections are also cast on the upper surface of the cast-iron plates to suit recesses in the lead plates. The cast-iron plates form a base on which is built a pile of lead plates in four tiers. Each of the three lower tiers is composed of fifteen blocks, 1 foot $7\frac{1}{4}$ inches by 11 inches by 3 inches in thickness, with projections on one side and recesses on the other side, $10\frac{1}{4}$ inches by 5 inches by 1 inch in depth, made to fit into one another and laid with even joints. The blocks in the top tier are made without projections; four of these are cast double, and one treble the size of the ordinary blocks. The double blocks are placed at the four corners, and the treble block is placed in the centre of the middle row, for the purpose of tying the blocks together. They are further secured from spreading by means of five transverse and three longitudinal wrought-iron straps intersecting at the centre of each block in the tier; at each intersection a 1-inch bolt passed through the straps and blocks holds the blocks securely down to the bed-plate. Great care was taken to have the blocks tightly wedged and fitted together into one mass, and up to the present they have not shown the slightest sign of movement.

Steel Cable.—The cable operating the counterweight was manufactured by Messrs. Felten and Guilleaume, of Germany, and is of galvanized crucible steel. It is $\frac{7}{8}$ inch in diameter, and is composed of six strands, each consisting of twelve wires 0·071 inch in diameter having a breaking-stress of 80 tons per square inch, and a hemp core. The cable weighs 1·1 lb. per foot, and is guaranteed to withstand a stress of 23 tons without breaking, whilst the working-stress is roughly about 1·5 ton. The cable is coated with a preservative composition to exclude moisture. At the buffer-trolley end the cable is permanently spliced to a thimble, whilst at the counterweight it is laid round a similar thimble, and the end is secured with three stout steel clips for the purpose of adjustment to allow for the stretching of the cable. This cable lasted until February 1906, when it was taken out and a new cable of the same size and manufacture was put in to replace it.

Hydraulic Buffer.—Hydraulic buffers have been in use in terminal railway-stations since about 1884, when they were first introduced at Liverpool Street and Fenchurch Street Stations. They formed the subject of a Paper by Mr. A. A. Langley, read before the Institution of Mechanical Engineers in 1886. The hydraulic buffer

adopted on the line under description has a stroke of 3 feet 6 inches, and is similar to those described by Mr. Langley; it is illustrated in Figs. 4 and 7, Plate 8. The cylinder is of cast-iron and is 10 inches in diameter and 4 feet in length. The metal of the cylinder is $1\frac{1}{2}$ inch in thickness for about three-quarters of its length from the front face, the thickness being increased to $1\frac{3}{4}$ inch for the last quarter. A rectangular parallel wrought-iron feather, turned on one side to fit the cylinder, is bolted on each side of the cylinder inside. The feathers are 3 inches in width by $1\frac{1}{2}$ inch in thickness at the back end, tapering to $\frac{1}{2}$ inch in thickness at the front end and running from cover to cover. The covers are each provided with two screwed glands; the inside gland, of cast iron, holds the hat-leathers in position, whilst the outer one, of brass, is packed with gasket packing. The hat-leathers have proved very effective in keeping the cylinder tight, with very little friction, and have worn well, the original pair being still in good order after two years of constant service. The packed glands are run slack, being intended only as a stand-by in the event of the hat-leathers being blown out or otherwise failing. The piston is a plain disk of cast iron, $9\frac{63}{64}$ inches in diameter, having rectangular waterways 3 inches in width by $1\frac{1}{2}$ inch in depth cut longitudinally on each side to work along the feathers in the cylinder. A rubber pad, $1\frac{1}{2}$ inch in thickness, bored out to clear the collar-nut on the piston, is inserted at the bottom end of the cylinder as a final cushion. The clearance between the piston and the cylinder is equal to an area of 0·25 square inch, and the full waterways have an area of 6 square inches. As the piston advances, the waterways are gradually diminished by the tapering feathers, causing gradual increase in the pressure behind the piston, which finally brings it to rest.

To bring the buffer forward again, after being pushed in by the counterweight, cast-iron weights are attached to a crosshead on the tail-rod of the piston by means of chains running over pulleys at the front end. The crosshead is provided with a pair of wheels running on a timber guide-way faced with angle-bars. The weights are in three sets, attached to the ends and centre of the crosshead respectively. The centre weight is divided into a main weight and a jockey-weight, the latter being suspended from a 6-inch pulley running in guides which rides on the centre chain. The main weight comes to rest on cross-timbers in the well, placed so that the jockey-weight is supported on the slack of the chain when the buffer is in the forward position. The jockey-weight is lifted by the chain before the main weight, and thus acts as a spring in taking up the first shock on the chain. The cylinder is automatically fed with water by gravitation

from a small tank fixed at the top end of the terminal pit (Figs. 4, Plate 8), and a brass non-return ball-valve is fitted on the pipe close to the cylinder. A small valve attached to the top end of the cylinder is provided for the escape of air which accumulates inside the cylinder, and which should be regularly released to prevent it from collecting and displacing the water to an extent that might prove disastrous. The consumption of water has been very small, owing to the excellent working of the hat-leathers, the meter not having yet registered 100 gallons.

The speed of the cars on the incline is usually not more than 5 or 6 miles per hour, and as a precaution against the cars being driven too fast over the top of the hill, where they leave the buffer-trolley, and where the reduced gradient tends to accelerate their speed, Nicholson Street is made a regular stopping-place. The cars stop while still in contact with the trolley, and the counterweight, having only a few feet to travel when the car is started again, does not get up an appreciable speed before it is brought to rest by the hydraulic buffer.

Testing.—The cylinder of the hydraulic buffer satisfactorily stood a test-pressure of 1,600 lbs. per square inch in the shops, with its covers on, and the piston-rod in position. Before the cover-plates were put on the terminal-pit, the counterweight was hauled up the incline to distances representing speeds of 1 to 12 miles per hour at the moment of impact with the hydraulic buffer. The trolley was released from the hauling-tackle by means of a trigger and was allowed to run freely in order to attain the required speeds. Up to about 4 miles per hour the rod seemed to be driven in without appreciable resistance, coming quietly to rest at the finish; but as the speeds increased the resistance became more noticeable. At the higher speeds the rod in every instance came to a dead slow about 9 inches from the end of its stroke, and then travelled quietly to the end, but on no occasion could any motion be detected in the cylinder or bed-timbers. At 10 miles per hour the sudden jar caused the side-weight chains to jump off their pulleys; and to prevent this a wrought-iron hood-guard was bolted over each pulley, after which they gave no more trouble even at the higher speeds. The jockey-weight on the centre chain proved very successful, the main weight rising in every instance without any apparent jerk.

The line was constructed by the Department with day-labour; the high rate of wages, and the minimum rate of 7*s.* per day for all labour in vogue at the time, brought the cost of the work up to a total of £5,606, including a contract let to the Clyde Engineering Company

for the manufacture of the hydraulic buffer, counterweight and buffer-trolleys, for £448.

The work was carried out under the direction of Mr. H. Deane, M.A., M. Inst. C.E., Engineer-in-Chief for Railway Construction, with the Author as Supervising Engineer for Tramway Construction, Mr. Thomas Rhodes being the Assistant Engineer on the works.

The Paper is accompanied by a sheet of drawings, from which Plate 8 has been prepared.

(*Paper No. 3600.*)

"On the Propulsive Power of Screws Necessary to Avoid Cavitation."

By JACQUES AUGUSTIN NORMAND, M. Inst. C.E.

ONE of the most important facts recently recorded in naval architecture is the very great influence of propulsive surface on the maximum speed of screw-vessels. Sir William White in his Presidential Address made special reference to it.

For instance, three English cruisers of the same class—"Good Hope," "Leviathan" and "Drake"—were tried in 1902. The hulls and the power of the engines were the same. The screws were alike in diameter and developed surface, with only a slight difference in pitch. The three cruisers attained very nearly the same speed—about 23 knots per hour with 31,000 I.HP.

Now the curves of slips and efficiencies showed that the slip increased, and the efficiency was reduced, at speeds exceeding about 20 knots per hour, the variations being so great at 23 knots per hour that this speed could not be exceeded without an enormous waste of power. There was every reason to think that cavitation came into play. Accordingly new screws of the same diameter, with nearly 40 per cent. increased surface, were fitted to the "Drake," and the result was an increase of more than 1 knot per hour with the same power. In the "Essex," of the "County" class, an addition of nearly 2 knots per hour is said to have been obtained simply by enlarging the screws.

The Author thinks it is possible to determine beforehand the necessary propulsive power of screws. In the suction-pipe of a centrifugal pump, for instance, the speed of the water cannot exceed a certain value, dependent on the atmospheric pressure and on the height of suction in relation to the water-level in the lower tank. When this speed is exceeded the column of water breaks. The great practical inferiority of jet-propellers can be only partly explained by the importance of slip; it is mostly due to the breaking

of the column of water in the suction-channel. Similarly, in a screw-vessel water rushes to the screws with a velocity dependent on the atmospheric pressure augmented by the height of water above the part of the screw considered. Should this speed exceed a certain value, the column of water is broken in front of the screws and cavitation takes place, although the water is not contained in a pipe as it is in pumps. Accordingly, the problem to be solved is— to find for a given vessel the propulsive power of screws such that the backwards velocity of water in front of them, at maximum power, shall not exceed that at which cavitation begins.

The following calculations are founded on the well-known laws—

(i) The resistance of a ship in motion is equal to the momentum of the water thrown backwards. It is accordingly proportional to the weight of water moved per second by the propellers multiplied by the acceleration given to it.

(ii) The resistance of a ship in motion is approximately proportional to

$$(\text{Displacement})^{\frac{2}{3}} \times (\text{speed of the ship})^2,$$

or, more exactly—since the product of resistance and speed varies as the I.HP.—to

$$\frac{\text{I.HP.}}{\text{Speed of the ship}}.$$

For ships of similar class, having about the same immersion of screws, let—

R denote the resistance of the ship at maximum speed.
D ,, ,, displacement of the ship in tons.
P ,, ,, maximum I.HP.
n ,, ,, number of screws.
Δ ,, ,, diameter of screws in feet.
S ,, ,, developed surface of all the screws in square feet.
r ,, ,, ratio of the total surface S to the sum of the surfaces of the circles of diameter Δ.
V ,, ,, maximum speed of the ship in knots per hour.
v ,, ,, backwards acceleration given to the water in front of the screws at maximum speed. When no cavitation takes place, v is the same in front of and behind the screws.
W ,, ,, weight of water set in motion by the propellers at maximum speed.

a, b, c, d, e, f denote the constants.

According to law (i) above

$$R = a\,W|v,$$

and since the backwards maximum speed v is a constant not to be exceeded,

$$R = b\,W \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

The weight of water set in motion is proportional to the number of screws, to the square of the diameters of the screws, to the speed of the ship—since the length of the liquid cylinders actuated per second varies nearly as the speed—and to a certain function of the ratio r.

The propulsive power of screws of a given diameter increases with the ratio r, but not so rapidly as the ratio itself increases, just as the turning-power of a rudder increases with the width of the rudder, but not so rapidly as the width. Assuming the function to be $r^{\frac{3}{4}}$ then

$$W = c\,n\,\Delta^2\,r^{\frac{3}{4}}\,V,$$

and since, according to formula (1), it must be proportional to the resistance R, by law (ii) above,

$$n\,\Delta^2\,r_{\frac{3}{4}}\,V = e\,D^{\frac{2}{3}}\,V^2,$$

or

$$n\,\Delta^2\,r^{\frac{3}{4}} = e\,D^{\frac{2}{3}}\,V \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and

$$n\,\Delta^2\,r^{\frac{3}{4}}\,V = f\frac{P}{V},$$

or

$$n\,\Delta^2\,r^{\frac{3}{4}} = f\,\frac{P}{V^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

$n\,\Delta^2\,r^{\frac{3}{4}}$ may be called the propulsive power (different from the developed surface) of the screws.

Formulas (2) and (3) may be interpreted thus: In order to avoid cavitation, the propulsive power of screws must be proportional to the product of the resistant surface of the ship into its speed, or more exactly to the ratio borne by the I.HP. to the square of the speed. The latter rule is the more exact, because, in estimating the speed to be attained with a given power, the fineness of the hull and all other causes affecting speed may be considered.

Assuming these rules to be true, it is easy to see why insufficiency in propulsive surface has been observed only recently.

The old rule, which is still given in works on Naval Architecture,

was to proportion the propulsive surface to the resisting surface $D^{\frac{2}{3}}$, whereas it ought to vary as $D^{\frac{2}{3}}V$, or to $\frac{P}{V^3}$, whereas it ought to vary as $\frac{P}{V^2}$. The maximum speed of cruisers, for instance, was about 20 knots per hour 20 years ago, against 24½ knots per hour to-day; so that this old rule, applied to the latest ships, gave a propulsive power of screws too small by

$$\frac{24\frac{1}{2} \text{ knots} - 20 \text{ knots}}{20 \text{ knots}} = 22\frac{1}{2} \text{ per cent.},$$

or for a given diameter of screw, a blade-surface too small by 32 per cent., since the propulsive power of a screw does not vary as rapidly as the surface of the blades.

That the propulsive power of screws ought to increase with the speed of the ship is proved by the fact that the sudden fall of efficiency and the abnormal increase of slip only occur at full power. A further proof is that in twin-screw torpedo-boats the consumption-trials at about half the maximum speed are often made with one screw only, although both screws may be barely sufficient at full speed to avoid cavitation.

The foregoing theory was proposed in 1899.[1] An Appendix to the Paper referred to gave the constants *e* and *f* for numerous vessels, the maximum speed of which ranged between 11½ knots and 23 knots per hour. Amongst these figured the "Powerful" and "Terrible," about which the Author then said:—

"It is much less dangerous to exaggerate the propulsive surface than to make it too small. The propulsive surface of some vessels in the annexed Table is considerable, and nevertheless their efficiency is very high. On the other hand, some vessels, the 'Powerful, for example, the surface-constant of which is very low, show a considerable fall of their curves of efficiency at maximum speed. It is probable that the propulsive surface of the sister-ship 'Terrible,' although superior to that of the 'Powerful,' is still insufficient."

And further on:

"There exist certainly many screw-vessels the maximum speed of which would be increased by 1 knot, in fine weather and with clean bottom, by an enlargement of the propellers."

The values of the constants *e* and *f* for some recent cruisers and battleships of the British Navy are given in the Appendix. The

[1] "Règles approximatives pour le calcul de la surface propulsive." Bulletin de l'Association technique maritime, vol. x.

data of the trials are for the greater part taken from the technical press. The Author trusts they are sufficiently accurate to show the minimum values of the constants which must be adopted for such vessels in order to avoid cavitation.

At maximum speed, the constant *f* (much more reliable than *e* for the reasons stated) must not be less than 6 or 7; and there are good reasons to think that higher values than 7 may be used with advantage. In the French cruiser "Château Renaud," at 24·18 knots per hour, *f* is 9·5, and in the old 13-knot or 14-knot battleships of Dupuy de Lôme, the efficiencies of which were very good, it is about 10.

It may be objected that when the draught of water is sufficient, screws of much smaller propulsive power may be used with steam-turbines. The reason for this difference does not seem to be unaccountable, but is not yet quite clear.

The importance of immersion is well known. Twelve years ago,[1] the Author showed that, in a torpedo-boat at moorings, an increase of 8 inches in the immersion of the axis of the propeller was sufficient to double the thrust at which cavitation began.

On the other hand, it is an undeniable fact that with a speed of rotation limited by the use of reciprocating engines, cavitation does take place, when the speed increases, if the propulsive power of the screws is insufficient. This Paper is limited to a consideration of the latter case.

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[1] See *Engineering*, vol. lviii, p. 58.

[APPENDIX.

APPENDIX.

		"Powerful."		"Terrible."		"Drake." Former Screws.		"Drake." Actual Screws.	"Essex." Former Screws.	"Essex." Actual Screws.	"Dominion."
Displacement in tons	D	14,200		14,200		14,100		14,100	9,800	9,800(?)	14,400
Number of screws	n	2		2		2		2	2	2	2
Diameter of screws in feet	Δ	19·50		19·50		19·17		19·17	16·00	15·75	17·50
Total developed surface in square feet	S	156		184		152		210	108	160	172
	$r = \frac{S}{0{\cdot}786\, n\, \Delta^2}$	0·261		0·308		0·263		0·363	0·268	0·410	0·357
Propulsive power of screws	$n\, \Delta^2\, r^{\frac{1}{3}}$	278		315		270		344	191	254	283
Speed in knots per hour	V	20·98	21·80	20·81	22·41	22·08	23·05	24·11	22·79	24·8	19·5
Slip, per cent.	..	12·4	18·6	14·4	15·7	13·9	18·0	13·3	..	..	..
I.HP.	P	18,460	25,900	18,500	25,648	23,103	30,557	31,400	22,219	22,219(?)	18,438
Efficiency	$\frac{V^3}{D^{\frac{2}{3}}\, P}$	293	235	286	257	272	234	260	244	314	260
	$e = \frac{n\, \Delta^2\, r^{\frac{1}{3}}}{D^{\frac{2}{3}}\, V}$	0·0226	0·0217	0·0256	0·0239	0·0209	0·0200	0·0244	0·0183	0·0223	0·0225
	$f = \frac{n\, \Delta^2\, r^{\frac{1}{3}}\, V^2}{P}$	6·64	5·10	7·36	6·16	5·71	4·73	6·37	4·45	7·02	5·84

(*Paper No. 3632.*)

"Note on the Cavitation of Screw-Propellers."

By SYDNEY WALKER BARNABY, M. Inst. C.E.

IN a Paper[1] by Sir John Thornycroft and the Author, read before the Institution in 1895, the phenomenon of "cavitation," which had been observed on the destroyer "Daring," was described. It was explained as being due to the limited speed with which the water could follow the blades of the screw, which caused cavities to be formed in the water forward of the screw, having no connection with the surface, and probably filled with aqueous vapour. It was shown that this resulted in a very serious loss of efficiency. From an analysis of the slip-curves of the screws of the "Daring" the Authors calculated that cavitation commenced when the thrust amounted to about 11¼ lbs. per square inch of projected blade-surface of the propellers at a depth of 1 foot below the surface.

The statements made in that Paper as to the probable influence of cavitation on the efficiency and design of propellers as speeds increased, met with much incredulity at the time, but they have been amply justified by events, and cavitation has become a well-known phenomenon in connection with high-speed vessels.

Shortly after the reading of the Paper referred to, the Hon. C. A. Parsons, M. Inst. C.E., carried out a series of experiments with a beautifully-devised apparatus consisting of a model screw-propeller working in a tank of water from the surface of which the pressure of the atmosphere had been removed by means of an air-pump, and was able to measure the thrust per unit area of surface at which cavitation commenced. The results which he obtained corroborated the Author's calculation of 11¼ lbs. per square inch mean pressure on the blades.[2] The Author has recently had an opportunity, through the kindness of Mr. Parsons, of witnessing experiments with the apparatus. The effects produced are striking and instructive. The

[1] "Torpedo-boat Destroyers." Minutes of Proceedings Inst. C.E., vol. cxxii, p. 51.

[2] See Transactions of the Institution of Naval Architects, 1897, p. 241.

small propeller, 2 inches in diameter, is revolved at about 1,200 revolutions per minute, and until the air-pressure on the surface of the water is reduced it works in quite a normal manner, no cavities being visible in the water. A few strokes of the air-pump, however, are sufficient to cause blisters to appear at the leading edge of the screw-blades and the after part of the boss. As the vacuum is increased these blisters spread over the propeller until the latter is almost hidden from sight, having the appearance of being inclosed in a silken bag from which streams of bubbles escape astern in beautiful spirals which persist for a considerable distance abaft the screw. This bag or cavity in which the propeller is enveloped when it is revolved at about 1,500 revolutions per minute is very unstable and can be seen to writhe and change its form, to break off and re-form, and although the tendency is no doubt aggravated by variation in the vacuum above the surface, it is easy to understand the serious vibration which always accompanies cavitation.

For a screw of any given pitch-ratio working at the rate of slip which corresponds with maximum efficiency, the thrust per unit of surface will be proportional to the square of the speed of the ship. This will be the case whatever size the ship may be, and is true whether the ship be propelled by one large screw or by many small screws.

That the thrust per unit area of blade-surface varies as the square of the speed, can be proved as follows:—With given pitch-ratio and efficiency the disk-area per horse-power is inversely proportional to the cube of the speed; and the total thrust of the screw for a given horse-power is inversely proportional to the speed. Therefore for double the speed there would be but one-eighth the total propelling-surface, and one-half the total thrust, and the thrust per unit of blade-surface must consequently be four times as great. (See Appendix.)

In a Paper published in the "Bulletin de l'association technique maritime," in 1899, Mr. J. A. Normand, M. Inst. C.E., referring to the variation in the rules given by different authorities for determining propulsive surface, observed:—"The existing uncertainty relates only to the function of the speed which should enter into the calculation of propulsive surface. Should this surface be independent of the speed, as in the original rules, or be proportional to the square of the speed, as affirmed by Messrs. Thornycroft and Barnaby, or should it be proportional to the speed simply as I propose?"

The Author believes that if Sir John Thornycroft and he were right in stating that cavitation would commence when a definite amount of thrust per square inch was exceeded, and if Mr. Parsons's

tank measurements were correct, then the propelling-surface must be proportional to the square of the speed and not to the speed simply. The Author has, up to the present time, seen no reason for doubting the correctness of the conclusions arrived at after the trials of the "Daring."

In a Paper on "The Screw-propeller," read before the Institution in 1890,[1] the Author gave a Table of propeller constants for disk-area and revolutions which he called C_A and C_R constants. This Table is reproduced in the Appendix, Table I. These constants were derived from the experiments of Mr. R. E. Froude and depended upon the equations—

$$\text{Disk-area} = C_A \times \frac{\text{I.HP.}}{(\text{speed in knots})^3} \text{ and}$$

$$\text{Revolutions} = C_R \times \frac{\text{speed in knots}}{\text{diameter in feet}}.$$

From the first of these equations, together with that for the total thrust of a screw, namely,

$$\text{Thrust} = \frac{\text{effective horse-power}}{\text{speed of ship} \times \text{thrust deduction-factor}},$$

it is possible to calculate the speed at which the thrust of any given propeller of the type used in the model experiment, and situated at the same depth below the water-surface as the screws of the "Daring," viz., 1 foot, will begin to be affected by cavitation. The calculations are given in the Appendix.

The general form of the equation is,

$$V_c^2 = \frac{C_A \times \text{thrust deduction} \times A_p \times T_c \times \text{blade correction}}{1 \cdot 125 \times (\text{wake correction})^3},$$

where V_c denotes the speed of the ship, in knots per hour, at which cavitation begins;

C_A the disk-area constant from Table I (Appendix),

A_p the value of the ratio $\frac{\text{projected area}}{\text{disk-area}}$,

T_c the thrust in lbs. per square inch of projected surface.

The blade-corrections and wake-corrections are given in the Appendix. The values of V_c calculated from this formula for propellers of different pitch-ratios and slip-ratios are given in Table II (Appendix).

[1] Minutes of Proceedings Inst. C.E., vol. cii, p. 74.

The following assumptions have been made in calculating these cavitating speeds:—

Thrust deduction = 10 per cent.

Propulsive coefficient = 0·5.

Projected area ratio ≒ that of a three-bladed propeller of the form used in the model experiments of Mr. Froude, which the Author calls "standard" area. This had elliptical blades of width =0·2 × diameter.

As speeds increase, cavitation can only be avoided by immersing the screw more deeply or by enlarging the blade area. It is not usually possible to increase the immersion, since this involves an increase in the draught of water of the ship, and the effect is very slight as the thrust can only be increased by $\frac{3}{8}$-lb. per square inch of surface for each extra foot of immersion. When a blade of "standard" area gives too small a surface to prevent cavitation, there are three ways in which the surface can be augmented, viz.:—

1. The blades may be made wider, other things remaining the same;

2. A lower abscissa value may be used, that is, the disk-area constant may be taken from the columns in Table I, to the left of column 9, which give maximum efficiency;

3. A larger pitch-ratio may be chosen, which will involve a larger diameter and a reduction in the rate of revolution. It will be seen by reference to Table II that increasing the pitch-ratio is less efficacious in raising the cavitating speed than reducing abscissa value. By raising the pitch-ratio, for example, from 1 to 1·5, the cavitating speed is raised at 9 abscissa value 1·9 knots, but a reduction in abscissa value at 1·0 pitch-ratio from the 11 column to the 9 column raises the cavitating speed 3·6 knots.

It is not to be supposed that cavitation imposes a limit to the speed which can be obtained by means of screws. Much higher speeds than any yet reached will doubtless be achieved, but the screw efficiency becomes less as speeds increase. It has already become necessary to make great changes in the form of blade, and to accept thrusts exceeding that at which cavitation is believed to commence, in vessels of the high speeds now attained.

Messrs. Thornycroft have used screws in their 30-knot destroyers having thrusts estimated at 13 lbs. per square inch and having very wide blades. It is difficult to say how much the propeller efficiency is reduced at this high thrust.

It will be seen that, under the conditions assumed in Table II, a screw of standard area designed to run at the slip corresponding

with maximum efficiency begins to cavitate at a speed varying between 23·0 knots per hour and 27·4 knots per hour—depending upon the pitch-ratio. If a thrust of 13 lbs. per square inch be accepted, and if also the blades be made extremely wide, so that, for example, the developed area = 0·74 disk-area, then the cavitating speeds can be raised very considerably, as shown in Table III (Appendix).

Here the speeds in column 9 range from 36 to 43 knots per hour. To attain these speeds multiple screws would almost certainly be required in order to admit of the employment of sufficiently fast-running engines, although the thrust per unit of surface at given abscissa value is the same, however many screws are used.

The cavitating speeds at lower abscissa values (see column 7) are shown to go up to more than 50 knots per hour, but the screws would require to run at such a relatively low speed of revolution that it would be very difficult to get machinery sufficiently light with the present design of marine engine to make such speeds attainable.

Examples are given in the Appendix of the use of the formula for cavitating speed.

[APPENDIX.

APPENDIX.

Let A denote disk-area in square feet,
P " I.HP. per screw,
V_c " speed of vessel, in knots per hour, at which cavitation commences,
C_A " constant for disk-area from Table I,

$$\text{Propulsive coefficient factor} = \frac{\text{propulsive coefficient}}{0 \cdot 5}.$$

$$\text{Then (1) } A = C_A \times \frac{P \times \text{propulsive coefficient factor} \times \text{blade correction}}{V^3 \times (\text{wake correction})^3}.$$

Let T_c = thrust in pounds per square inch of projected surface,

$$A_p = \text{value of ratio } \frac{\text{projected area}}{\text{disk-area}},$$

$$\text{Then (2) } V_c = \frac{P \times \text{propulsive coefficient} \times 33{,}000}{T_c \times 101 \cdot 33 \times \text{thrust deduction} \times A \times A_p \times 144}.$$

Substituting the value of A from (1) and dividing by V,

$$V_c^2 = \frac{T_c \times \text{thrust deduction} \times A_p \times C_A \times \text{blade correction}}{1 \cdot 125 \times (\text{wake correction})^3}.$$

Taking $T_c = 11 \cdot 25$,
Thrust deduction for 10 per cent. wake $= 0 \cdot 9$,
$A_p = 0 \cdot 207$ for $1 \cdot 2$ pitch ratio,
Blade correction for a three-bladed screw of standard area $= 1 \cdot 155$,
Wake correction for 10 per cent. wake = unity,

Then $V_c^2 = C_A \times 2 \cdot 155$.

$$V_c = \frac{\sqrt{C_A}}{0 \cdot 683}.$$

NOTE.—In Table III the blade-correction for a three-bladed screw having a developed area $= 0 \cdot 74$ disk-area is taken as $0 \cdot 87$.

Example.—To prove that thrust per unit of surface varies as the square of the speed.

1. Find the disk-area of a screw of 1·0 pitch-ratio to work at maximum efficiency. Speed of vessel 12 knots per hour; I.HP. 6,000. Ratio of projected blade-area to disk-area to be 0·220 to 1. Ten per cent. wake.

The disk-area constant from Table I to suit these conditions is 251.[1]

$$\text{Disk-area} = 251 \times \frac{\text{I.HP.} \times \text{blade correction}}{V^3} = 1{,}008 \text{ square feet.}$$

Projected blade-area = 221 square feet.

Thrust in lbs. per square inch of projected area

$$= \frac{0 \cdot 5 \times 6{,}000 \times 33{,}000}{12 \times 101 \cdot 33 \times 0 \cdot 9 \times 221 \times 144} = 2 \cdot 83 \text{ lbs.}$$

[1] For method of using this Table see the Author's Paper on "The Screw-propeller," Minutes of Proceedings Inst. C.E., vol. cii, p. 74.

2. Find the disk-area of a screw of the same model and same abscissa-value for a vessel of the same power, but having a speed of 24 knots per hour.

$$\text{Disk-area} = \frac{251 \times 6{,}000 \times 1 \cdot 155}{24^{3}} = 126 \cdot 2 \text{ square feet.}$$

Projected area $= 27 \cdot 8$ square feet.

Thrust in lbs. per square inch of projected area

$$= \frac{0 \cdot 5 \times 6{,}000 \times 33{,}000}{24 \times 101 \cdot 33 \times 0 \cdot 9 \times 27 \cdot 8 \times 144} = 11 \cdot 25 \text{ lbs.}$$

That is *four times* the thrust per square inch of the screw for 12 knots per hour.

Example of the use of the formula for cavitating speed.

Find the cavitating speed of a three-bladed screw of standard area, of 1·0 pitch-ratio, 9 abscissa-value, 10 per cent. thrust deduction, 10 per cent. wake, 1·155 blade correction, and 1 foot immersion of blade tips. Thrust per square inch to be 11·25 lbs.

$$V_c^{2} = \frac{11 \cdot 25 \times 0 \cdot 9 \times 0 \cdot 22 \times 251 \times 1 \cdot 155}{1 \cdot 125} = 5{,}760.$$

$\therefore V_c = 24$ knots per hour, which is the speed given in Table II for these conditions.

TABLE I.

Pitch Ratio.	Efficiency per Cent. 67		69		69		68		66		63	
	C_A	C_R	C_A	C_R	C_A	C_R	C_A	C_R	C_A	C_R	C_A	C_R
0·8	304	128	215	134	157	142	115	150	86	160	65	171
0·9	329	114	234	120	170	127	125	135	93	144	71	154
1·0	355	104	251	109	184	115	135	123	100	131	76	140
1·1	380	95	270	100	196	105	144	113	107	120	82	128
1·2	405	87	288	92	210	97	154	104	115	111	87	119
1·3	431	81	306	85	224	91	163	97	122	103	93	111
1·4	456	76	325	80	236	85	173	90	129	97	98	104
1·5	482	71	342	75	250	79	183	85	136	91	104	98
1·6	507	67	360	71	263	75	193	80	144	87	109	93
1·7	533	63	378	67	276	71	202	76	151	82	115	88
1·8	558	60	396	64	290	68	212	73	159	78	120	84
1·9	584	57	415	61	304	65	222	69	166	75	125	81
2·0	609	55	432	58	315	62	231	67	173	72	131	77
2·1	635	52	450	56	329	59	241	64	180	69	136	75
2·2	660	50	469	54	342	57	250	62	187	67	142	72
2·3	685	48	486	52	355	55	260	59	194	64	148	69
2·4	710	47	505	50	369	53	270	57	202	62	153	67
2·5	736	45	523	48	381	52	280	56	209	60	159	65
	7		9		11		13		15		17	
	Abscissa values.											

$$\text{Disk area} = C_A \times \frac{\text{I.HP.}}{(\text{speed in knots per hour})^3}. \qquad \text{Revolutions} = C_R \times \frac{\text{speed in knots}}{\text{diameter in feet}}.$$

TABLE II.—SHOWING SPEEDS AT WHICH CAVITATION COMMENCES WITH THREE-BLADED SCREWS OF STANDARD AREA.

Pitch Ratio.	Efficiency per Cent.						A_p.
	67	69	69	68	66	63	
	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	
0·8	27·3	23·0	19·6	16·7	14·5	12·6	0·232
0·9	27·9	23·5	20·0	17·1	14·8	12·9	0·226
1·0	28·4	24·0	20·4	17·5	15·1	13·2	0·220
1·1	29·0	24·5	20·8	17·8	15·4	13·5	0·213
1·2	29·5	24·9	21·2	18·1	15·7	13·7	0·207
1·3	29·9	25·3	21·5	18·4	15·9	13·9	0·200
1·4	30·3	25·6	21·8	18·6	16·1	14·1	0·194
1·5	30·7	25·9	22·1	18·8	16·3	14·2	0·188
1·6	31·0	26·2	22·3	19·0	16·5	14·3	0·181
1·7	31·3	26·4	22·5	19·2	16·6	14·4	0·176
1·8	31·5	26·6	22·7	19·3	16·7	14·5	0·170
1·9	31·7	26·8	22·8	19·4	16·8	14·6	0·165
2·0	31·9	26·9	22·9	19·5	16·9	14·7	0·159
2·1	32·0	27·0	23·0	19·6	17·0	14·8	0·155
2·2	32·1	27·1	23·1	19·7	17·1	14·9	0·150
2·3	32·2	27·2	23·2	19·8	17·2	15·0	0·145
2·4	32·3	27·3	23·3	19·9	17·3	15·0	0·142
2·5	32·4	27·4	23·4	20·0	17·3	15·0	0·137
	7	9	11	13	15	17	
	Abscissa values.						

$$A_p = \frac{\text{Projected area}}{\text{Disk-area}}.$$

TABLE III.—SHOWING CAVITATING-SPEEDS OF THREE-BLADED SCREWS, ASSUMING 13 LBS. THRUST PER SQUARE INCH AND DEVELOPED AREA = 0·74 DISK-AREA.

Pitch Ratio.	Efficiency per Cent.						A_p.
	67	69	69	68	66	63	
	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	Knots per Hour.	
0·8	42·7	36·1	30·7	26·1	22·7	19·7	0·652
0·9	43·6	36·8	31·4	26·7	23·1	20·2	0·635
1·0	44·5	37·5	32·0	27·3	23·6	20·7	0·620
1·1	45·4	38·2	32·6	27·8	24·1	21·1	0·600
1·2	46·2	38·9	33·2	28·3	24·5	21·4	0·582
1·3	46·8	39·5	33·7	28·7	24·9	21·7	0·563
1·4	47·4	40·0	34·2	29·0	25·2	22·0	0·546
1·5	48·0	40·5	34·6	29·4	25·5	22·2	0·530
1·6	48·5	40·9	35·0	29·7	25·8	22·4	0·510
1·7	49·0	41·3	35·3	30·0	26·0	22·5	0·495
1·8	49·3	41·6	35·5	30·2	26·2	22·7	0·478
1·9	49·6	41·9	35·7	30·3	26·3	22·8	0·465
2·0	49·9	42·1	35·8	30·5	26·4	23·0	0·447
2·1	50·1	42·2	36·0	30·7	26·6	23·1	0·436
2·2	50·2	42·4	36·2	30·8	26·7	23·3	0·422
2·3	50·3	42·5	36·3	31·0	26·9	23·4	0·407
2·4	50·5	42·7	36·5	31·2	27·0	23·5	0·400
2·5	50·6	42·8	36·6	31·4	27·0	23·5	0·385
	7	9	11	13	15	17	
	Abscissa values.						

$$A_p = \frac{\text{Projected area}}{\text{Disk-area}}.$$

(*Paper No. 3594.*)

"The Flow of Underground Water."

By William Ralph Baldwin-Wiseman, M.Sc., Assoc. M. Inst. C.E.

During the past three years the Author has endeavoured to investigate the relationship between the porosity of a rock and the flow of water through its interstices, under various pressures.

The research resolved itself into four main divisions, three of which were experimental and the fourth statistical, namely:—

1. Experiments on the rate of flow of water through moderately large blocks of stone, of various thicknesses, under pressures varying between atmospheric pressure and 75 lbs. per square inch above atmospheric pressure.
2. Experiments on the variation of hydraulic pressure within a rock at various depths from the pressed surface.
3. Experiments on the relative porosity and retentivity of rock and sand.
4. An investigation of the statistics of pumping- and filtration-plants.

Since 1838, when Stephenson [1] first noted the existence of a cone of depletion in the draining of the Kilsby tunnel, the rapid increase of population at home and the constantly increasing demand for irrigation abroad have caused probably more investigators to be attracted to hydrology than to any other branch of engineering. Amongst others, Bland,[2] in 1832, first instituted systematic well-soundings, and prepared a section on which he ruled a hydraulic gradient. Darcy,[3] in 1856, stated that the velocity of flow of ground-water was proportional to the slope of the hydraulic gradient, and varied for the same head inversely as the thickness of the strata penetrated. Roberts,[4] in 1861, demonstrated that the flow of water through sandstone varied with the pressure (Table I (6), Appendix), but he obtained some erroneous values as to porosity. About the same time Ansted attempted to measure flows in chalk under low pres-

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[1] W. Boyd Dawkins, "James Forrest" lecture. Minutes of Proceedings Inst. C.E., vol. cxxxiv, p. 254; and D. T. Ansted, "Water and Water-supply," p. 33. London, 1878.

[2] W. Bland, Philosophical Magazine, vol. xi, p. 88. London, 1832.

[3] H. Darcy, "Les fontaines publiques de la ville de Dijon." Paris, 1856.

[4] Isaac Roberts, Proceedings of the Liverpool Geological Society, 1868–69.

sures (Table I (7), Appendix), but unfortunately the methods chosen were open to some doubt, and liable to experimental error. Hagen,[1] in 1869, questioned the variation of flow with pressure, but agreed with Darcy that the flow varied inversely as the thickness of the strata traversed. Theim,[2] in 1870, first studied the motions of underground water, and was followed by Fournier,[3] Frühling,[4] Kroeber, Lueger,[5] Mancini,[6] Oesten, Smreker, Soyka, Thévenet, and others. Their work has been further amplified by the investigations of the hydrology of various districts by the United States irrigation officers and various civil engineers and commissions at home and abroad; whilst Prestwich,[7] Boyd Dawkins, Whitaker and other geologists have largely contributed to the question of hydrogeology.

According to Seelheim,[8] who also investigated the variation of pressure with porosity in clays and sand, the flow varies faster than the pressure producing it; whilst, according to Hazen,[9] who has done much work on the relative sizes of sand-grains and the effect of the size of the grains upon the flow, the velocity of flow is more nearly proportional to the square root of the head. Fuertes,[10] in a continuation of this work, has shown that a small variation in the size of the grain materially affects the velocity-coefficient, whilst a considerable variation of the radius of the cone of depletion, where it cuts the underground water-surface, has but little effect; he has also shown that the drop of the water-level when pumping has a greater effect upon the yield than the diameter of the well.

Of the more purely physical investigations, Cohen,[11] in 1892, showed that the viscosity of water varied inversely as the pressure, and thus confirmed Seelheim's statement that the flow increased faster than the pressure; whilst Thorpe and Rodgers,[12] in 1894, showed that the

[1] Hagen, "Handbuch der Wasserbaukunst." Berlin, 1869.

[2] A. Theim, "Verfahren für Messung natürlicher Grundwassergeschwindigkeiten." Polytechnisches Notizblatt, vol. xlii, p. 229. Mainz, 1887.

[3] E. Fournier, "Sur la Vitesse des Eaux souterraines." Also "Sur la Propagation des Eaux souterraines." Bulletin de la Société Belge de Géologie, de Paléontologie et d'Hydrologie, vol. xvii. Brussels, 1903.

[4] Frühling, "Handbuch der Ingenieurwissenschaften."

[5] O. Lueger, "Wasserversorgung der Städte."

[6] F. Mancini, Annales des Ponts et Chaussées, June, 1890, p. 823. Also extract, Minutes of Proceedings Inst. C.E., vol. ciii, 1890-91, p. 436.

[7] J. Prestwich, "A Geological Enquiry respecting the Water-bearing Strata round London." London, 1851. Also, "On the Geological Conditions affecting the Water Supply to Houses and Towns." Oxford, 1876.

[8] Seelheim, Zeitschrift für analytische Chemie, vol. xix. Weisbaden, 1880.

[9] A. Hazen, Report of the Massachusetts State Board of Health, 1892.

[10] J. H. Fuertes, Engineering Record, vol. xxxix. New York, 10 Dec., 1898.

[11] R. Cohen, Annalen der Physik und Chemie, vol. xlv. Leipzig, 1892.

[12] Thorpe and Rodgers, Philosophical Transactions of the Royal Society, A, pt. 1, 1894.

viscosity varied inversely as the temperature. King,[1] following Graham and Meyer's deductions that Poseuille's law holds for water as well as for gas, measured with an aspirator the rate of flow of gases, at moderate pressures, through stones and sand, and deduced from these experiments a corresponding flow of water. Girard [2] showed that the velocity of flow of water in capillaries varied with the temperature and with the nature and amount of the dissolved salt; and also that the temperature effect was more marked in saline waters than in fresh waters (Table I (3), Appendix). Slichter,[3] from observations on a well at the University of Wisconsin, stated, in contradiction of Seelheim, that the flow did not increase so rapidly as the pressure. He assumed a rock-mass to be an aggregation of spheres arranged in various degrees of compactness, the least density being obtained when the tangent-planes were the faces of a circumscribing cube, giving a porosity of 47·64 per cent.; and the greatest density when the tangent-planes were the faces of a rhombohedron of angle 60°, giving a porosity of 25·95 per cent. Continuing with this treatment he deduced the length, and the greatest and least cross-sectional area of the interstitial tube, for a given thickness of rock, and calculated the time of flow of water through such tubes. Lee [4] has stated that the rate of flow of water through the pores of a rock in the immediate vicinity of a well is greater than at points more remote, the rate diminishing more quickly with a small well than with a large one.

Several formulæ for the flow of water in capillaries have been advanced from time to time, the more important of which are given in Table I (1, 2, 4, 5, 8), (Appendix).

The stones selected for experiment are described in Table II (Appendix). They are not all typical water-bearing rocks, but were chosen as representative of fairly porous rocks of the different formations in the considerable geological range from the Carboniferous to the Cretaceous. Each stone, upon being received from the quarry, rough-dressed to its true bed, was carefully inspected for flaws, fractures, iron spots, etc., and was dressed to a cylinder 13 inches in diameter and 6 inches in height. At the conclusion of the first set of experiments the height was reduced to 3 inches by dressing, and at

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[1] F. H. King, "Principles and conditions of the movement of ground-water." 19th Annual Report U.S.A. Geological Survey. Washington, 1897–98.

[2] Girard. M. Jamin, "Cours de physique de l'école polytechnique." Paris, 1891. Vol. i, pt. 2, p. 124.

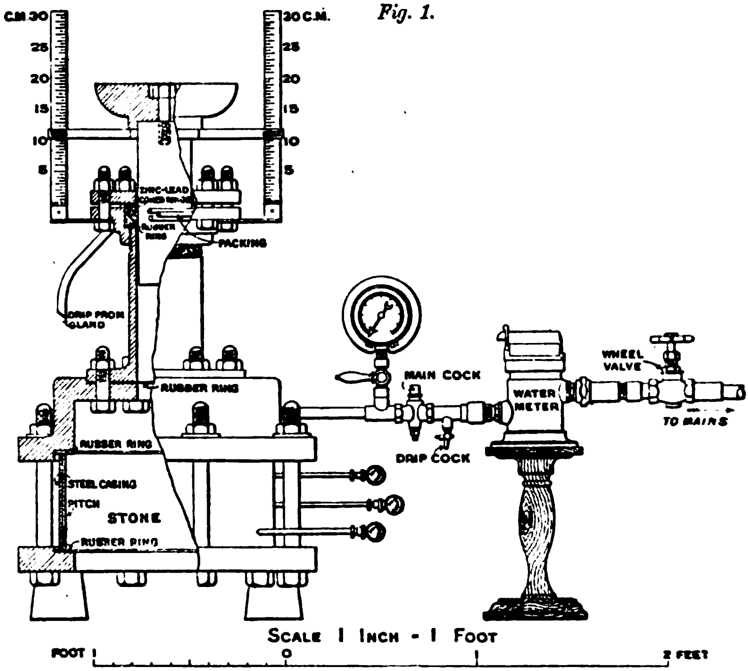
[3] C. S. Slichter, "Mathematical investigation of ground-water movements." 19th Annual Report U.S.A. Geological Survey. Washington, 1897–98.

[4] Willis Lee, "Underground Water of Gila Valley, U.S.A." Water Supply Irrigation and Bulletins. U.S.A. Geological Survey. Washington, 1905. Series O, No. 104, p. 17.

the conclusion of the second set of experiments the stone was sawn into strips or broken up with a hammer; the pieces were carefully inspected for flaws, but none were found.

In making a set of experiments a stone cylinder was encased in a steel ring having an internal diameter of 13¼ inches, and the space between the ring and the stone was filled with boiling pitch, poured in in small quantities and pressed down, to prevent side-flow from the stone cylinder. The encased stone was then placed on a ring of rubber sheeting, resting in a groove, formed in a heavy cast-iron

Fig. 1.



HYDRAULIC ACCUMULATOR USED IN THE EXPERIMENTS.

base-plate, having a circular hole, exactly 1 foot in diameter, in the centre. A hydraulic accumulator was placed over the stone, a water-tight joint being effected by a ring of rubber sheeting, and was bolted down to the base-plate; the rubber rings, pressing heavily above and below the stone, prevented the ingress or egress of water except over the area of a 12-inch circle. The accumulator, designed by the Author for this work, and shown in *Fig. 1*,[1] consisted of a dish-shaped reservoir, accurately turned, where it fitted the

[1] See also "The Puddling Effect of Water flowing through Concrete." Minutes of Proceedings Inst. C.E. vol. clxiii, p. 320.

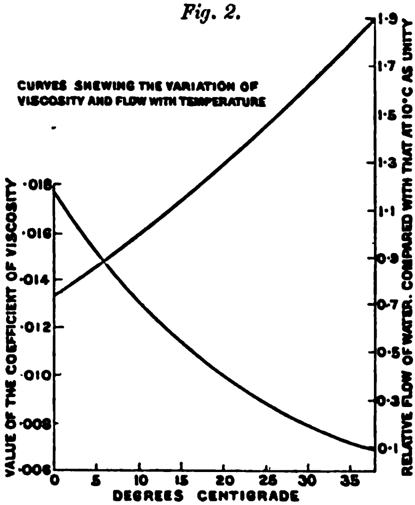
stone, to a circle of 1 foot diameter, above which was mounted a cast-iron sleeve, carrying a gland, through which an accurately-turned loaded steel plunger, of slightly less diameter than the interior of the sleeve, was made to slide; this gland was at first fitted with leather collars, but these proved useless at low pressures, and two zinc-lead rings, coned internally and fitted with three rings of packing between them, were substituted; this packing proved efficient at both high and low pressures. A hole was tapped into the base of the gland, and a drip-cock was inserted, the water from which was collected in a measuring-glass and deducted, together with the leakage from the drip-cock of the main cock, from the calculated discharge, at the conclusion of each experiment. The plunger carried a double pointer, the ends of which moved over two steel scales rigidly fixed, one on either side of the piston, to the flange on the upper end of the sleeve. The quantity of water which passed into the stone was measured by the drop of the piston, after deducting the leakage, during the interval considered; often for months together, there was no leakage, and the gauged leakage rarely if ever exceeded 1 per cent. of the calculated flow. The discharge from the under-side of the stone fell upon a large zinc tray, from which it drained to a receptacle; the quantity of water collected in this way during any experiment was compared with the calculated discharge, and although, as might be expected, these quantities were not identical, the approximation was sufficiently close to furnish an excellent rough check on the work.

The apparatus was so proportioned that a drop of the piston of 1 centimetre was equivalent to a discharge of 62·06 cubic centimetres; and the net area exposed was such that a discharge of 1 cubic centimetre per second was equivalent to a discharge of 1 gallon per hour per square foot of surface. The stone having been fitted into the apparatus, the main cock was closed and the drip-cock opened, until the pump was clear of air and a steady stream of water issued from it; the drip-cock was then closed and the main cock opened, and water was sent into the apparatus until the pointers on the plunger had risen to a fixed point some distance above that at which readings were to be commenced, any air in the apparatus being expelled at the gland; the main cock having been closed and the drip-cock opened, the plunger was then loaded with a moderate load and was run several times unrecorded in order to thoroughly charge the stone with water. Starting with a low pressure, readings were taken of the positions of the pointers at minute intervals and of the gauge-pressure on a Bourdon gauge fixed on a ½-inch pipe, the centre of which was exactly 1 inch above the upper surface of the stone; the temperature of the air and of the water, the load on the plunger

and the leakage from the drip-cock and gland were also recorded for each experiment, and the density of the effluent was observed during the first few experiments upon each stone. The calculated discharges from which the leakages, if any, were deducted, were corrected for viscosity [1] to the equivalent flow at 10° C. according to the curve of flow (*Fig. 2*), but no corrections were made for the variation of compressibility of water [2] with pressure and temperature, since the range of these, namely, 1 to 6 atmospheres and 6° C. to 17° C., was so small as to affect the results but slightly, in the fourth place of decimals.

The flow through the Daresbury (Lower Keuper) sandstones (Triassic strata) was so considerable as to preclude the use of the pump in supplying fresh water to the apparatus; the inlet-pipe was therefore connected, as shown in *Fig. 1*, to the town mains, the pressure on which varied between 55 lbs. and 62 lbs. per square inch during the day and between 72 lbs. and 78 lbs. per square inch during the evening. The plunger, loaded with weights, was maintained at a certain height by adjusting the inflow by means of a wheel-valve. The experiment was commenced about a minute after stable conditions of flow and pressure were attained and was continued for an hour, or, in the case of a drop of pressure taking place in the mains, till the instant the plunger changed its position, the experiment being in each case terminated by closing the main cock on the inlet-pipe. The inflow was measured by passing it through a meter and was checked against the discharge, which was collected in gauged receptacles; and in the case of an incomplete run, the time was recorded in minutes and seconds.

Fig. 2.



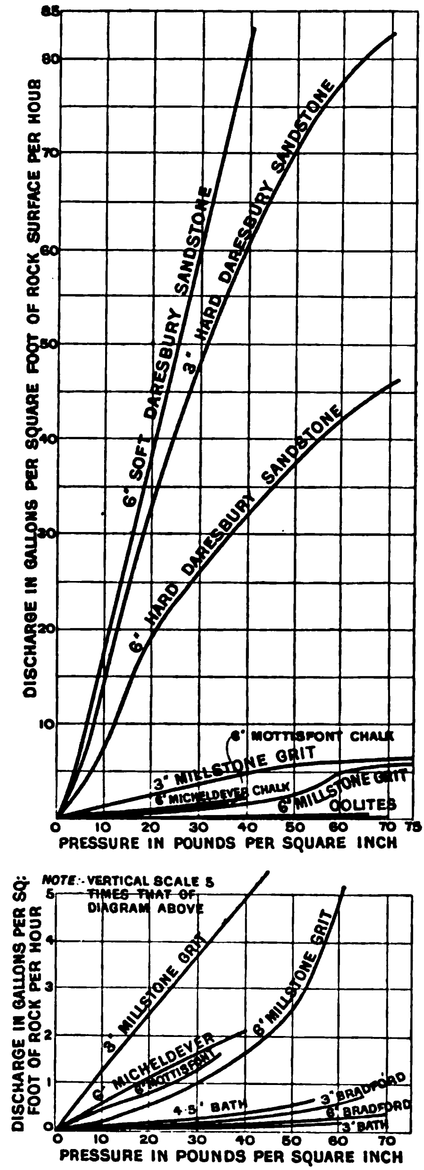
The records given in Tables III and IV (Appendix) are not continued in all cases to the maximum pressure of 75 lbs. per square

[1] Osborne Reynolds, Philosophical Transactions of the Royal Society, pt. 1, 1886; R. Cohen, Annalen der Physik und Chemie, vol. xlv. Leipzig, 1892; Thorpe and Rodgers, Philosophical Transactions of the Royal Society, A, pt. 1. 1894.

[2] Grassi, Annales de chimie et de physique, 3e série. Paris, 1851; and Pagliani and Vincentini, Nuovo Cimento, 1884; also P. G. Tait, "Properties of Matter," 4th edition, p. 198. London, 1899.

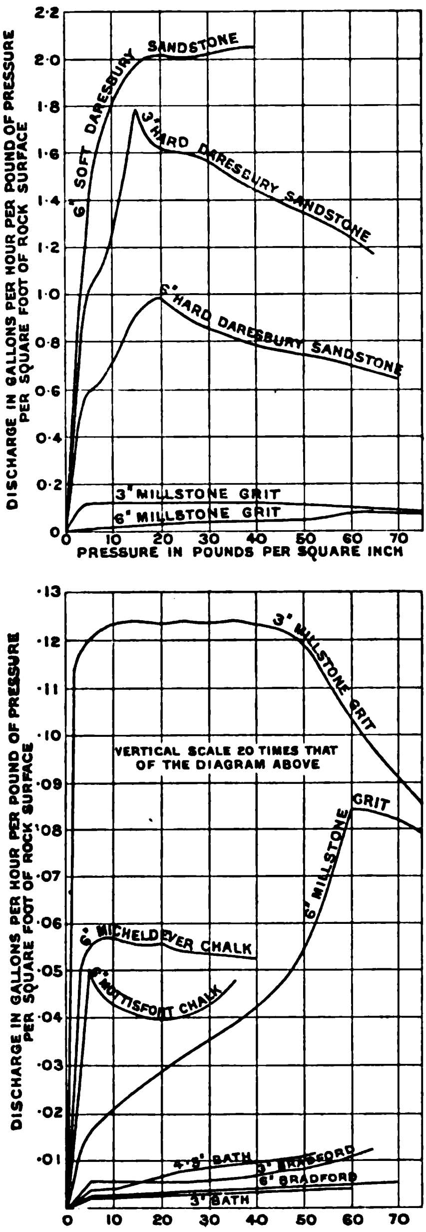
inch; the chalks flaked on the underside at pressures slightly in excess of those to which the records are carried, and at still higher pressures cracked across and let the water through in a large body; the oolites and the hard Daresbury stone were crushed slightly at the edges at pressures exceeding those recorded, and the flow through the soft Daresbury stone was so great as to be very difficult to deal with at higher pressures. The laboratory was free from vibration and was maintained at a fairly constant temperature throughout the whole research. The flows, reduced to an equivalent flow at 10° C., are given in Table III (Appendix), and these, divided by the respective pressures producing them, are detailed in Table IV (Appendix). These results are also shown graphically in *Figs. 3* and *4*. From an inspection of the diagrams it is at once evident that the flow does not vary in any direct manner with the pressure, diameter of grain, or porosity, through any considerable range of pressure. With the Bradford and Bath oolites, excepting the 3-inch Bradford between the pressures of 5 lbs. and 25 lbs. per square inch, the discharge increases continually with the

Figs. 3.



25 lbs. per square inch, the discharge increases continually with the

Figs. 4.



pressure, but with the chalk a peculiar phenomenon occurs: with the Mottisfont chalk there is a steady decrease of flow per unit pressure between 5 lbs. and 20 lbs. per square inch, succeeded by a steady increase between 20 lbs. per square inch and the pressure under which it failed; with the Micheldever chalk there is a small increase between 5 lbs. and 10 lbs. per square inch, followed by a decrease between 10 lbs. and 15 lbs. per square inch, succeeded by another small increase between 15 lbs. and 20 lbs. per square inch, which in turn gives place to a continuous decrease until the load of rupture is reached, *Figs. 4.* The probable explanation of this phenomenon is that the flow increases with the pressure up to a critical velocity, beyond which any increase of pressure is expended in eddies and in the enlargement of the pores by rupturing the septa between contiguous pores; with an enlarged cross-section the flow might once more become steady, and continue so until the increasing pressure once more gave rise to a critical velocity. With the harder rocks, however, no such oscillatory variations were

observable within the range of the pressures covered by the experiments; but the flows increased faster than the pressures up to a certain load and thenceforward decreased continuously, the change from increase to decrease occurring at a lower pressure for a thin than for a thick section of stone. It also appears on examination of the Tables that for relatively thin sections the flow does not vary exactly in the inverse ratio of the thickness, and, as in the case of the 3-inch Bath weatherstone, the problem is in all probability complicated by the loss of energy in eddies; nevertheless, for moderate differences of pressure in considerable thicknesses of rock, the flow will most probably vary inversely as the thickness, for the same head, as deduced by Darcy and others. The flows given in Table III (Appendix), although comparable as to the volume of the flow, are not comparable as to the velocity of the water-particles through the stones, to obtain which, the relative porosities of these stones must be taken into consideration. In Table V (Appendix) the Author has determined the total pore-space of each test-piece, and has styled the number of times this must be charged and discharged in order to pass 1 gallon of water "the discharge-factor"; this factor, multiplied by the several discharges at any one pressure (in Table V the pressure of 5 lbs. per square inch is chosen), affords a measure of the relative velocities of the water-particles in the several stones, under that pressure. In order to determine the effect of dissolved matter in the water upon the discharge, the Author conducted two sets of experiments, with fresh water and with brine, the results of which are given in Table IIIA (Appendix). At the conclusion of the set of fresh-water experiments upon the 4½-inch Bath oolite the apparatus was emptied and strong brine was pumped in. The density of the brine was determined by drawing off some of it from the drip-cock just before the commencement and after the end of each experiment; the percolating water was collected in a clean dish placed on the drip-tin, and its density was also determined for each experiment. Brine at constant density was pumped in upon four occasions, but on the fifth and subsequent occasions fresh water was supplied to imitate under experimental conditions the case of rain-water obtaining access to the zone of a coastal pumping-station which has been overpumped during a drought. Having regard to the wide divergence of the results of Girard's experiments on the flow of solutions through capillary tubes (Table I (3), Appendix), the somewhat erratic results of the Author's experiments are not surprising, especially when it is remembered that, for the uniform diameter and smooth walls of a capillary tube there was substituted

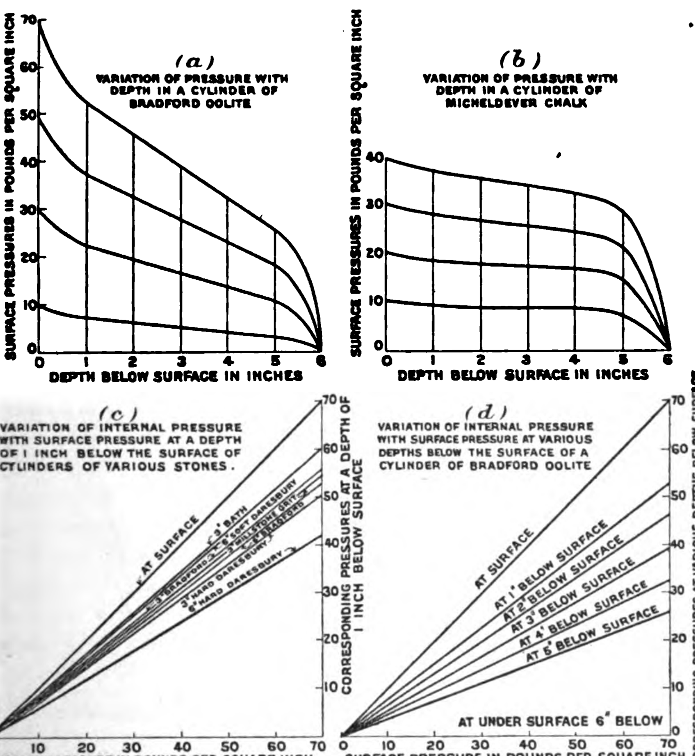
the constantly varying diameter, roughness, and sinuosity of an interstitial tube such as a water-particle traverses in its course through a rock mass. In experiments Nos. 1 to 4 (Table IIIA, Appendix) brine was pumped into the apparatus at constant strength; in Nos. 5 to 9 fresh water was pumped in, reducing the density of the brine above the stone, with the intention of steadily reducing the salinity of the inflowing water, but the variations were so considerable and inconsistent that only the first five readings are given; the brine was then entirely drawn off, the apparatus was well flushed, the stone was cleaned, and fresh hot water was pumped into the apparatus to flush out the pores, but again the records were variable and inconsistent, and only the first two readings are given. One conclusion which stands out strongly is that once a salt has obtained access to the pores of a rock, its removal is a matter of extreme difficulty, and that it is most judicious to cease pumping immediately signs of brackishness are manifested. The water used throughout the experiments was obtained from the Southampton Corporation mains. It is an excellent water, pumped from the chalk at Otterbourne, and having an initial hardness of about 18°, which is, however, reduced to 6° before delivery. Owing, however, to some rearrangement of the plant at the pumping-station, it was delivered unsoftened for a few days, whilst the Author was experimenting with a 3-inch section of millstone grit; these records are omitted from the Tables given, but it was noted that for similar pressures the flow was less, and for a continuance of the same pressure for a day or more, the flows slightly diminished with time, probably on account of a small deposition of calcium carbonate in the pores, reducing the cross-sectional area of the pores, or affecting the surface-tension, and inferentially the flow.

For determining the variation of internal pressure in the stone, but little modification of the apparatus was required. Holes, 2½ inches in depth, were tapped radially into the side of the cylinder, the centres of the holes being an exact number of inches below the surface of the stone, and separated from each other by arcs of circles, 4 inches in length, measured horizontally on the surface of the steel casing; into these holes copper tubes, 9 inches in length and 0·115 inch in internal diameter, were fitted, making a water-tight joint with the casing, and connected at their outer ends by water-tight joints to small but accurate pressure-gauges. The plunger was loaded, and the gauges were observed until the lowermost gauge indicated a constant pressure in place of a steadily increasing one; readings were then taken upon all the gauges at intervals of 5 minutes for 2 hours or more. The mean results of several dozen independent experiments are given in Tables VI and VII (Appendix), and

the diagrams in *Figs.* *5* and *6* have been prepared from these records.

In general the variations were similar for all the rocks, with the exception of the chalk, although differing in magnitude. In *Fig.* *5* (*a*) plotted from the variations of pressure at several points within a 6-inch block of Bradford oolite, but typical of all the others, except the chalk *Fig.* *5* (*b*), it is seen that the greatest variation of pressure occurs in the immediate vicinity of the surfaces, whilst within the mass of the rock the variation is more uniform and relatively

Figs. *5.*



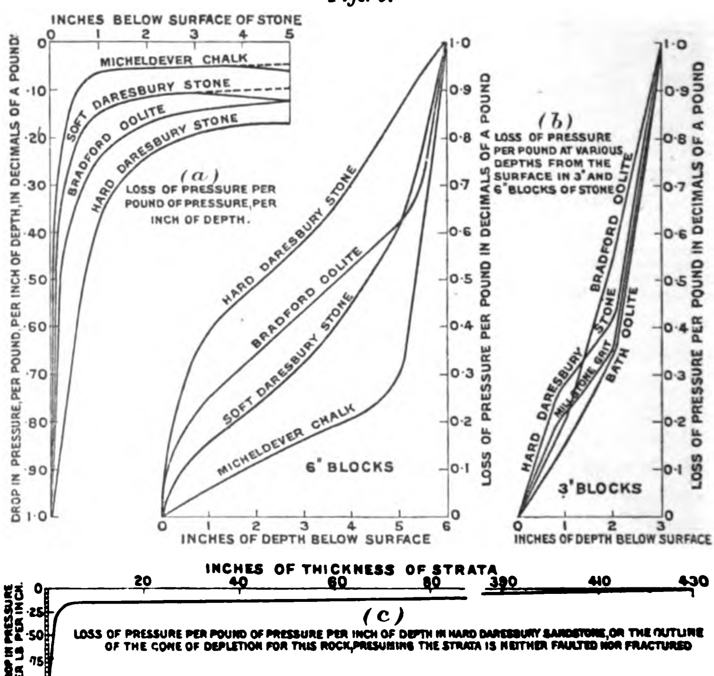
small. From an examination of *Fig.* *5* (*d*) it is evident that for the same rock the loss of pressure at any given depth is directly proportional to the surface pressure. Thus, in the case of the 6-inch Bradford oolite, the loss of pressure in the first inch of depth below the surface is 24·6 per cent. of the total pressure of 70 lbs. per square inch, and 25·5 per cent. of the total pressure of 20 lbs. per

square inch, similar agreement occurring at other depths as shown in the following Table:—

BRADFORD OOLITE, 6 INCHES IN THICKNESS.

	Loss of Pressure per Inch Depth below the Surface, expressed as a Percentage of the Surface Pressure	
	At 70 Lbs. per Square Inch.	At 20 Lbs. per Square Inch.
	Per Cent.	Per Cent.
Loss of pressure between surface and 1 inch below surface	24·6	25·5
Between 1 in. and 2 ins. below surface .	9·4	10·0
,, 2 ins. ,, 3 ,, ,, .	9·3	9·0
,, 3 ,, ,, 4 ,, ,, .	9·7	9·5
,, 4 ,, ,, 5 ,, ,, .	9·4	9·0
,, 5 ,, ,, 6 ,, ,, .	37·6	37·0

Figs. 6.



In *Figs. 6* are plotted curves showing the loss of pressure in 6-inch and 3-inch blocks of stone, and also the loss of pressure per unit pressure per inch of depth at each inch of depth from the surface;

in this diagram (*Fig. 6* (*a*)), neglecting the drop of pressure in the 4th and 5th inches of the soft Daresbury stone, and in the 5th inch of the Micheldever chalk, due to the proximity of the underside, there is obtained a series of curves which continue with gradually diminishing inclinations as the thickness of the strata traversed increases. If any one of these curves be carried forward, by averaging or otherwise, as shown for the hard Daresbury stone in *Fig. 6* (*c*), it cuts the datum-line in a point more or less remote from the origin (in the case selected the point is at a distance of 430 inches from the origin). At this point the loss of pressure is zero, or the effect upon the flow, at this distance, of a difference of pressure of 1 lb. per square inch is negligible. If the vertical scale be made to represent *n* lbs. instead of 1 lb., the scaled distance multiplied by *n* will represent the distance at which *n* lbs. will have no appreciable effect upon the flow; or, in other words, the curve presents for the strata upon which the experiments are conducted a true scale outline of a vertical section of the cone of depletion set up by the pumping producing the difference of pressure, provided the strata in the vicinity of the well be neither faulted nor fractured, although the presence of a fault or fracture will be a disturbing factor only in proportion to its magnitude. The application of this method is illustrated by the following Table, deduced from the pressure experiments :—

Strata.	Loss of Pressure of 1 lb. per Square Inch in a distance of	Pressure Corresponding to Cone of Depletion of			
		¼ Mile Radius.		½ Mile Radius.	
	Inches. Feet.	Lbs. per Sq. Inch.	Feet of Water.	Lbs. per Sq. Inch.	Feet of Water.
Hard Daresbury stone	430 = 35·8	36·9	85·2	73·7	170·3
Bradford oolite . .	960 = 80·0	16·5	38·1	33·0	76·2

The percentage interference of two wells within the area of each other's action can also be determined graphically by this method, by drawing the two cones of depletion and evaluating the relative areas of the cone of one well, with and without the other's action.

The losses of pressure in the Daresbury sandstones, the millstone grit, and the oolites, are more or less comparable, as is shown by *Figs. 5* (*c*) and *6* (*a* and *b*); but in the chalk (*Fig. 5* (*b*)) the drop of pressure in the first inch is small compared with that in the sandstones, whilst the loss of pressure in the last inch is of much greater magnitude, being almost double that of the drop of pressure in the last inch of the Bradford oolite, *Fig. 5* (*a*). For purposes of com-

parison the losses of pressure at each inch of depth in the 6-inch Bradford oolite and the 6-inch Micheldever chalk under a surface-pressure of 40 lbs. per square inch, are given in the following Table :—

MICHELDEVER CHALK AND BRADFORD OOLITE, 6 INCHES IN THICKNESS.

	Expressed as a Percentage of the Surface Pressure	
	For Chalk.	For Oolite.
	Per Cent.	Per Cent.
Loss of pressure between surface and 1 inch below surface	7·0	25·0
Between 1 in. and 2 ins. below surface	3·7	9·5
,, 2 ins. ,, 3 ,, ,,	5·3	9·2
,, 3 ,, ,, 4 ,, ,,	5·2	9·5
,, 4 ,, ,, 5 ,, ,,	8·0	9·2
,, 5 ,, ,, 6 ,, ,,	70·8	37·6

From this Table it appears that water experiences little difficulty in obtaining ingress to chalk but considerable resistance to its egress. Disregarding the effect of cushioning air, which only operates when chalk is more or less thoroughly depleted of its water, the capillarity of the chalk appears to accelerate the inflow of water and to impede its discharge, although it assists in the desiccation of the chalk by evaporation when the chalk is more or less fully charged with water. These points are still further illustrated in Tables VIII, VIIIA and VIIIB (Appendix).

Various investigators[1] have from time to time, for many years past, determined the amount of contained water in different rocks when fully charged, but it is only within comparatively recent years that any particular attention has been paid to resoakage, and yet this question of the rate and amount of recharging of depleted strata after a long-continued drought, or within the cone of depletion of a well, is of the greatest importance in all waterworks problems. In 1895–96 Professor W. Boyd Dawkins made some experiments on resoaking, and found that whilst after 114 days 2·542 gallons of water were evaporated per cubic foot of chalk, only 2·437 gallons per cubic foot, or 95·86 per cent of that previously evaporated, were absorbed; and since, as he

[1] Royal Commission Report, Building stone for Houses of Parliament 1839; Royal Commission Report, Rivers Pollution, 1868; British Assoc. Report, 1882; Ansted, Minutes of Proceedings Inst. C.E., vol. ix, p. 368; Cowper, Minutes of Proceedings Inst. C.E., vol. xlvii; Boyd Dawkins, Minutes of Proceedings Inst C.E. vol. cxxxiv; J. Roberts, Proceedings of the Liverpool Geological Society, 1868–69; C. C. Moore, Proceedings of the Liverpool Geological Society, 1901–03, parts 2 and 3; Seelheim, Zeitschrift für analytische Chemie, vol. xix, p. 387; F. H. King, "Principles and conditions of the movement of underground water." 19th Annual Report U.S.A. Geological Survey, 1897–98; Bischoff, "Chemical Geology," vol. i; E. J. Chapman, Philosophical Magazine, ser. 4, vol. vi, p. 118.

pointed out in the "James Forrest" Lecture[1] in 1898, this 2·542 gallons per cubic foot did not accurately represent the full water-capacity of the natural rock, the quantity reabsorbed becomes a relatively smaller percentage of the total water-capacity of the rock.

For the purposes of the Author's investigation of the effect of the contained air on the resoakage of a piece of rock, pieces of soaked stone were broken from the cylinder of stone in the apparatus, immediately after the conclusion of the flow experiments, when the pores were fully charged with water. By this method a water-content was obtained more closely approximating to natural conditions than that of ordinarily soaked specimens. Thus the two upper chalk specimens, from Mottisfont and Micheldever, contained 2·86 and 2·92 gallons per cubic foot respectively, an increase of 14·4 and 16·8 per cent. respectively on the customary 2·5 gallons per cubic foot. The specimens were weighed immediately after breaking them from the cylinder, and were air-dried for 6 or 7 days (Table VIIIB Appendix), the weight being noted at the end of the first half-hour, at the end of each succeeding hour for 8 hours, and thereafter daily; they were then maintained, in a steam-oven, at a temperature of 95° and 97° C., varying with the barometric conditions, until there was no appreciable decrease, for some considerable period, in the weight of the stone when cooled to the normal temperature; the difference in weight, dry and soaked, divided by the volume, accurately determined by one or two methods, gave the unit porosity recorded in the Tables, and this multiplied by one hundred gave the percentage porosity. In one experiment the Author repeatedly dipped a prism of dried chalk 3·82 inches by 1·53 inches by 1·26 inches in a bath of cold water for an instant only, and weighed it after each immersion; from the first till the eighth immersion the surface became dry in appearance immediately upon the removal from the water, but with each succeeding immersion after the eighth the surface retained a moist appearance for intervals which rapidly increased in duration. These experiments are recorded in Table VIII (Appendix), the amount of absorption from the commencement being expressed as a percentage of the volume of the stone, and also as a percentage of the total porosity, and the absorption at each interval being expressed as the thickness of a film enveloping the whole surface; thus for the

2nd	immersion	the	thickness	of this	film is	0·537	that	for the	first	immersion.
4th	"	"	"	"	"	0·261	"	"	"	"
6th	"	"	"	"	"	0·158	"	"	"	"
8th	"	"	"	"	"	0·081	"	"	"	"
10th	"	"	"	"	"	0·044	"	"	"	"
12th	"	"	"	"	"	0·008	"	"	"	"

[1] "On the Relation of Geology to Engineering." Minutes of Proceedings Inst. C.E., vol. cxxxiv, p. 254.

Any evaporation of the adherent water which may have taken place can hardly have affected the results, as the specimen was not weighed until the surface had a dry appearance; and the thickness of the film for any immersion is calculated from the actual difference of weight for that and the preceding immersion. The piece was also soaked for many days afterwards under 1 foot 6 inches depth of water, but did not regain its full quantity of contained water. The Author had intended to continue this experiment for an indefinite period, but an accident to the specimen unfortunately terminated the series of readings. The results of trials of two different methods of resoaking stone which had once been thoroughly dried are recorded in Table VIIIA (Appendix); with ordinary soakage in a tank under a head of about a foot of water the greater quantity of water was taken up in the first half-hour, and from examination of the previous Table it might be concluded that the greater portion of this had been absorbed in the first half-minute. From the results for Micheldever chalk given in Tables VIII and VIIIA (Appendix) it will be found that the absorption in 12 seconds' immersion is 35·96 per cent. of the total volume of the stone, whilst for an immersion lasting ½ hour this absorption is increased by only 10·57 per cent., the corresponding increments for 4½ hours and 47 hours being 12·01 and 17·85 per cent. respectively; and the latter percentages would in all probability have been considerably diminished had the 12 seconds' immersion been continuous instead of intermittent. Again, considering only the records given in Table VIIIA (Appendix), the increased absorptions for Mottisfont and Micheldever chalk at the end of 4½ hours are respectively 2·08 and 1·31 per cent. of the absorption at the end of the first half-hour, the corresponding increases after 47 hours' soaking being only 18·98 and 6·59 per cent. In the second series of soakage experiments, pieces of chalk were placed quite dry in a shallow glass vessel and were covered with water to a depth of ¾ inch. The vessel was placed under the receiver of an air-pump, which was steadily worked for 10 minutes; air was then admitted to the receiver, water was added to the vessel to restore the head to ¾ inch, and the receiver was again exhausted. At first the volume of air given off from the stone was considerable; but with each subsequent operation the volume of air given off steadily diminished, and towards the end of the first 1¼ hour it was almost inappreciable; during the next ¼ hour no air-bubbles were observable, and the amount of absorbed water at the end of this period was greater in the case of the Micheldever chalk, and as great in the case of the Mottisfont chalk, as that after 4½ hours'

ordinary immersion, proving conclusively the obstructive action of the air in the pores.[1]

In no case in the experiments undertaken did the volume of absorbed water, upon resoakage, equal the original quantity evaporated, although in one or two cases it approximated closely thereto, being 95 to 97 per cent. of the original volume; but it is quite probable that under the pressures which obtain in deep-seated strata, and given sufficient time, the interstices would be eventually fully recharged. Not only does the air in the interstices offer resistance to the inflow of water, but once in the rock mass it will cause or tend to cause instability of water-level,[2] since it is much more susceptible to variations of pressure and temperature than the interstitial water. It is also shown by Table VIIIB (Appendix) that the rate of evaporation rapidly diminishes with time; thus in the first 24 hours 15·2 per cent. of the total amount of contained water is evaporated, in the first 119 hours 26·2 per cent., and in the first 166 hours only 40·7 per cent., so that capillarity, by retarding outflow, to a certain extent diminishes the amount of water evaporated. The effect of the contained air, also, in the Author's opinion, to a great extent invalidates those researches which base the rate of flow of water in rock on that of air, as the interstitial air would not cushion the entrant air, but the whole would have a uniform pressure throughout the pores, whilst with water containing air, and with air in the pores of the rock, the pressure, and consequently the rate of flow, would vary from point to point.

Several formulæ for the flow of water in sand and rock have been deduced from the behaviour of water in sand, but it is necessary to know more of the previous treatment of the particular sand

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[1] Memoirs, Geological Survey of England and Wales. W. Whitaker, Geology of London, vol. i, p. 513.

[2] Baldwin Latham, British Association Reports, 1881, p. 614; 1883, p. 495.

O. Lueger, "Einfluss des Atmosphärendrucks auf die Ergiebigkeit von Brunnen und Quellen." Centralblatt d. Bauverwaltung, 1882.

C. S. Slichter, "Motions of Underground Waters." Water-Supply and Irrigation Bulletins, No. 67. Washington, 1902, p. 71.

F. H. King, Weather Bulletin, No. 5, 1892, p. 50. U.S.A. Department of Agriculture.

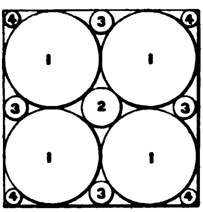
K. Honda, "Experiments on effect of tidal level and barometric pressure on underground-water levels at Tokio," described in *Water*, vol. 7, No. 78. London, June 15, 1905, p. 238.

M. F. Weyde, Abstract from la Revue Scientifique, "On the variation of underground-water level with barometric pressure," recorded in *Water*, vol. 6, No. 64. London, April 15, 1904, p. 151.

D. Pantanelli, "Variazioni sul Livello delle Acque sotterranee di Modena." Bollettino della Socièta geologica Italiana, vol. xvi, p. 319. Rome, 1897.

experimented upon before accepting the formulæ, for the porosity to a great extent depends upon the mere fortuity of the packing; thus, considering a simple mathematical case, if a cube of unit side be filled with eight spheres, No. 1, as shown in *Fig. 7*, the porosity will be 47·75 per cent., but if the space between these be filled by sphere No. 2, the porosity is reduced to 47·15 per cent If, again, smaller spheres, No. 3, be fitted in the positions indicated, there is a further reduction to 44·48 per cent., and finally, if the still smaller spheres, No. 4, be fitted in the remaining spaces, the porosity is ultimately reduced to 43·20 per cent. This percentage porosity may be still further reduced if other series of still smaller spheres be similarly arranged; and if mathematical precision be replaced by disorder, an almost endless variety of percentage porosities can be obtained with the same mass of material. In further illustration of this variability of porosity the Author conducted a series of experiments the results of which are given in Tables VIIIc and VIIId (Appendix). The filter-sand used was first dried by spreading it in thin layers upon an iron tray maintained at a temperature of about 150° C.; it was then carefully sifted in small quantities and poured from a height of 2 feet into the measuring-flask, the measurement being recorded as the volume "dry loose." With steady shaking for about ½ hour this volume was reduced by 12 to 15 per cent., and is recorded as the volume "shaken dry"; and this had a certain porosity recorded in the Table. But when the sand was agitated with water in a large beaker and drained until there was no flow whatever from the beaker, even when the sand in it was stirred, the contained water was 7·7 per cent. to 14·2 per cent. greater, probably because in the latter case each sand-grain was completely enveloped in a film of water, whereas in the former case the grains, being in more intimate contact with one another, were only partially enveloped by a film of water. This subject is further illustrated by reference to the statistics of filter-beds given in Tables XI and XII (Appendix). Taking into consideration the great pressure of superincumbent material during, and subsequent to, the deposition of any stratum, and the subsequent metamorphism induced by faulting, folding, intrusions, percolating water and other tectonic and erosive agencies, it is evident that the somewhat ephemeral relationships which may have held for the sand become altogether non-existent in, say, sandstone, and a formula based upon such

Fig. 7.



investigations becomes so restricted in its application as to be of little practical value. No formula, for instance, could automatically allow for a variation of 26·9 per cent. in the porosity of a Cheshire sandstone (Keuper) within a range of 8 yards from a fault, nor of a difference of 10 feet in the water-level of two cottage-wells in the Hertfordshire chalk separated merely by the width of a country lane; yet these are only examples chosen haphazard from many which might be cited. As each stratum considered in small local areas will have its own coefficients of roughness, size of grain, angularity, porosity and other controlling factors, and as the flow for any one rock does not vary in an absolute ratio with the pressure through any considerable range, it will be necessary to conduct experiments similar to those detailed in this research for each stratum and for each local variation of that stratum; a formula such as that given below, based upon such experimental knowledge and data derived from existing installations in the same stratum and locality, and in the hands of one well acquainted with the geological structure of the district, will furnish a fairly accurate estimate of the yield of a proposed well or the increased yield of an extension of an existing well. Such a formula, of course, will not automatically allow for the increased yield due to the proximity of a river, or to a well cutting a line of fissure, or an unpuddled fault-plane.

Let k be a transmission constant, varying with the strata, determined from existing data;
l the difference in feet between the water-levels when pumping and at rest;
h the height in feet of the rest-level above the base of the well or bore;
V the capacity of the well and bore at rest-level, in cubic feet;
t the thickness of the test-piece in feet, this being not less than 0·5 foot;
r the basal radius of the pressure-zone in feet for a difference of 1 lb. of pressure (loss of pressure experiments);
x the discharge in gallons per square foot per hour from the test-piece, at the pressure corresponding to the available head l in feet of water (flow experiments);
and y the yield of the well in gallons per hour for a reduction of level of l feet.

Then $$y = \frac{k\,x\,t}{l\,r}\sqrt{4\,\pi\,h\,V}.$$

In amplification of the formula the Author has calculated the yield of the St. Helens wells in the Bunter Beds (Triassic strata) upon the constants derived from the experiments with hard Daresbury sandstone (Lower Keuper sandstones, Triassic strata), as the nearest allied rock used in the experiments; the results are given in the following Table:—

No. 1 is Eccleston Hill Pumping-Station.
No. 2 „ Whiston „ „
No. 3 „ Knowsley „ „
No. 4 „ Kirkby „ „

No.	k	t in Feet.	r in Feet.	x in Gallons per Square Foot per Hour.	h in Feet.	l in Feet.	Volume at Reduced Level in Cubic Feet.	Actual Pumping.		Calculated Pumping Gallons per Hour.	Percentage Difference.
								Cubic Feet per Second.	Gallons per Hour.		
1	1,380	0·5	35·8	23·6	248	60	2,791	1·051	23,600	22,400	− 5·3
2	„	„	„	4·0	267	14	2,142	0·839	18,800	14,800	−27·0
3	„	„	„	27·8	609	75	9,152	1·867	41,900	59,700	+29·8
4	„	„	„	31·5	480	90	12,270	2·799	62,900	58,000	− 8·4

In conclusion, the Author desires to express his indebtedness to those engineers who generously supplied him with the details of the installations under their charge, from which Tables IX and XI (Appendix) were compiled; to Mr. Councillor Haysom, of Messrs. Garret and Haysom, Monumental Masons, Southampton, for carefully dressing the stones for the research, and to Mr. R. Askwith, M. Inst. C.E., for kindly supplying the Author with samples of filter-sand.

The Paper is accompanied by seven tracings, from which the Figures in the text have been prepared; and by the following Appendix.

[APPENDIX.

APPENDIX.

Table I.

(1) *Flow in Capillaries.*—M. Jamin, "Cours de Physique de l'école polytechnique." 4th ed. vol. 1, pt. 2, p. 124, Paris, 1891.

$$Q = 1836\cdot724\,(1 + 0\cdot0336793\,t + 0\cdot0002209936\ \ ^2)\,\frac{h\,d^4}{\ }$$

Q = flow in cubic millimetres per second.

t = temperature in degrees Centigrade.

h = head in millimetres of mercury at 10° C.

d = internal diameter of tube in millimetres.

l = length of tube in millimetres.

(2) *Osborne Reynolds's Formula.*[1]—Loss of energy per lb. of fluid passing through a pipe, length l feet, diameter d feet, with a velocity of v feet per second (in feet-units)

$$= \frac{l\,b^n\,p^{2-n}\,d^{n-3}\,v^n}{a}.$$

n for velocities below the critical velocity is 1.

" above " " 1·7 to 2·0.

$$v_{\text{critical}} = 0\cdot039\,\frac{p}{d}.$$

$$p = 1 \div (1 + 0\cdot0336793\,t + 0\cdot0002209936\,t^2).$$

$$a = 1\cdot917 \times 10^6.$$

$$b = 36\cdot8.$$

t = temperature in degrees Centigrade.

[1] Philosophical Transactions of the Royal Society, 1883 and 1896. J. Perry. "Applied Mechanics," p. 84. London, 1901.

TABLE I—*continued.*

(3) *Girard's experiments*[1] on the time of flow of the same volume of various solutions across a given length of tube under the same pressure.

Solution.	Strength.	Temperature. °C.	Time of Flow across Tube	
			In Seconds.	As Ratios.
Water	..	0	1,036	1·143
	..	60	906	1·000
Solution of common salt . . .	$\frac{1}{8}$	3	1,337	1·475
		60	443	0·489
,, potassium nitrate .	$\frac{1}{8}$	0	681	0·752
		60	310	0·342
,, sulphate of soda . .	$\frac{1}{8}$	20	872	0·962
		65	371	0·409

(4) *C. S. Slichter.*[2]—Formula for the flow through a column of soil.

$$q = (9 \cdot 3036 - 10) \frac{p\, d^2\, s}{\mu\, h\, k}.$$

q = cubic feet of water transmitted by a column of soil per minute.

p = difference of pressure at two ends of column in feet of water at 4° C.

s = area of cross-section of column in square feet.

h = height of column in feet.

μ = coefficient of viscosity of fluid.

k = constant varying with size of grain, porosity and μ.

d = diameter of soil grain in millimetres.

Formula for the flow in cubic feet per minute from a well.

(*a*) Neglecting resistance due to pipe friction $Q = \dfrac{2 \pi h k a}{\log_e \left(1 + \dfrac{600}{r}\right)}$.

(*b*) Considering resistance due to pipe friction and that offered by the water-bearing strata $Q = q \left(\sqrt{1 + \dfrac{q^2}{4 Q_0^2}} - \dfrac{q}{2 Q_0} \right)$.

[1] M. Jamin, "Cours de Physique de l'école polytechnique," vol. 1, pt. 2, p. 124. Paris, 1891.

[2] "Theoretical Investigation of the Motion of Ground Waters," 19th Annual Report, U.S. Geol. Survey, pt. 2, 1899, p. 295.

TABLE I—*continued.*

h = drop of water-level in feet when pumping.

r = radius of well in feet.

a = thickness of water-bearing strata penetrated in feet

k = transmission constant for that strata.

Q = discharge in cubic feet per minute.

q = estimated free discharge of water through well and casing under head h.

Q_0 = theoretical yield of well under head h if friction in head and pipe be neglected.

(5) *D. H. Fuertes.*[1]—Formula for yield of a well.

$$Q = 3 \cdot 142 \, P \, V \, (H^2 - h^2) \div \text{nat log} \left(\frac{2 \, R}{d}\right).$$

Q = yield of well in gallons per second.

P = percentage void. 30 to 40 for gravel. 20 to 40 for sand.

V = velocity coefficient = $c\,S^2$, where S is the effective size of the sand grain in millimetres, a uniform temperature of 50° F. being assumed.

H = height in feet from bottom of well to natural ground water-level.

h = height in feet from bottom of well to reduced water-level when pumping.

R = distance from well to approximate limit of zone of depression in feet.

d = diameter of well in feet.

(6) *Isaac Roberts.*—Proceedings, Liverpool Geological Society, 1868–69.

The flow of water through a block of sandstone 10½ inches thick under a pressure of 10 lbs. per square inch = 4·5 gallons per hour.

20 lbs.	,,	=	7·5	,,
46 lbs.	,,	=	19·0	,,

[1] "Yield of Wells and Galleries," *Engineering Record*, New York. Vol. 39, p. 28. Dec. 10, 1898.

TABLE I.—*continued.*

(7) *D. T. Ansted.*[1] The flow of water through 4 inches of chalk.
Under head of 2 feet of water = 0·428 gallon per day.
" " 6 " " = 1·138 " "

(8) *Allen Hazen.* *Formula* for velocity of flow through sand.

$$(a)\ v = cd^2 \frac{h}{l}(0\cdot70 + 0\cdot03\,t).$$ [2]

$$(b)\ v = cd^2 \frac{h}{l}\left(\frac{t + 10}{60}\right).$$ [3]

v = velocity at which a column of water of cross-sectional area equal to that of the sand-bed passes through it in metres per day.

d = effective size of sand grain in millimetres.

h = head lost in passing through sand at a velocity v in feet.

= thickness of bed in feet.

t = temperature of water in ° F.

c is an experimental constant varying with the condition of cleanliness of the sand.

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[1] Minutes of Proceedings Inst. C.E., cxxxiv, p. 261.
[2] Report Massachusetts State Board of Health, 1892, p. 541.
[3] W. H. Burr, "Ancient and Modern Engineering." New York, 1902, p. 302.

TABLE II.—TABLE OF DETAILS OF THE ROCKS USED IN THE EXPERIMENTS.

Rock.	Geological Formation.	Quarry and Locality.	Colour and Other Characteristics.	Weight in Lbs. per Cubic Foot.	Specific Gravity.	Mean Unit Pore-Space.	Contained Water in Gallons per Cubic Foot.	Mean Diameter of Particles in Inches.
Mottisfont Chalk	Upper Chalk . . .	Mottisfont Abbey, near Romsey, Hants, some distance below surface	White; fairly free from fossils and flints .	100·7	1·611	0·4586	2·862	0·0088
Micheldever „	„ „ . . .	E. bank at the S. end of the L.S.W.R. Co. tunnel near Micheldever, Hants, about 100 feet below surface	White; fairly free from fossils and flints .	96·6	1·545	0·4679	2·919	0·0058
Bath Weather-stone . . .	Great Oolite . . .	Bath Stone Firm's Quarries, Bath . .	Creamy; characteristic oolitic structure	133·3	2·133	0·2418	1·509	0·0110
Bradford Oolite	„ „ . . .	Winsley Quarries, Bradford-on-Avon .	Creamy-brown; characteristic oolitic structure . . .	114·8	1·836	0·1902	1·187	0·0087
Hard Daresbury sandstone .	Trias; Lower Keuper	J. and H. Ryder's Quarries, Daresbury, Cheshire	Chocolate-red; fine grained and compact	166·9	2·670	0·2519	1·572	0·0120
Soft Daresbury sandstone .	„ „ „	J. and H. Ryder's Quarries, Daresbury, Cheshire	Lighter red; somewhat friable . .	127·2	2·035	0·2315	1·444	0·0119
Millstone Grit .	Carboniferous; "rough rock"	B. Whitaker & Son's Quarries, Horsforth, Leeds	Creamy-grey; moderately small grained	131·4	2·103	0·1498	0·935	0·0289[1]

[1] The mean diameter in the case of the Millstone Grit refers to the finer material only. Some of the coarser fragments had diameters ranging from 0·2 inch to 0·3 inch.

TABLE III.—TABLE SHOWING THE VARIATION OF FLOW OF WATER WITH PRESSURE THROUGH VARIOUS ROCKS.

Flow expressed in Gallons per Hour per Square Foot of Exposed Surface.

Surface Pressure in			Chalk.		Bath Weatherstone.		Bradford Oolite.		Daresbury Sandstone.			Millstone Grit.	
			Mottisfont.	Micheldever.					Hard.		Soft.		
Lbs. per Square Inch.	Feet of Water.	Atmospheres.	Thickness in Inches 6	Thickness in Inches 6	Thickness in Inches 4·5	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 6	Thickness in Inches 3
5	11·59	0·34	0·247	0·277	0·018	0·009	0·010	0·025	3·00	5·11	7·10	0·079	0·603
10	23·08	0·68	0·430	0·564	0·036	0·018	0·020	0·049	7·30	12·62	18·12	0·197	1·235
15	34·61	1·02	0·613	0·830	0·071	0·028	0·032	0·072	14·00	26·71	29·74	0·370	1·858
20	46·15	1·36	0·795	1·117	0·125	0·041	0·050	0·096	19·75	32·64	40·21	0·567	2·471
25	57·69	1·70	1·010	1·341	0·179	0·055	0·070	0·121	22·98	40·07	50·19	0·779	3·093
30	69·23	2·04	1·268	1·596	0·232	0·072	0·093	0·154	26·25	48·13	60·79	1·023	3·706
35	80·77	2·38	1·655	1·851	0·295	0·092	0·118	0·197	29·19	54·89	71·50	1·322	4·300
40	92·31	2·72	..	2·085	0·366	0·114	0·145	0·243	32·10	60·02	81·75	1·668	4·904
45	103·85	3·06	..	..	0·429	0·138	0·173	0·339	34·91	65·64	..	2·046	5·479
50	115·38	3·40	..	..	0·527	0·165	0·202	0·391	37·53	70·16	..	2·613	5·913
55	126·92	3·74	..	..	0·643	0·194	0·233	0·488	40·12	74·21	..	3·580	6·111
60	138·46	4·08	..	..	..	0·222	0·267	0·606	42·25	78·14	..	5·037	6·195
65	150·00	4·42	..	..	..	..	0·304	0·760	44·10	80·86	..	5·430	6·252
70	161·54	4·76	..	..	..	..	0·343	..	45·67	82·56	..	5·682	6·336
75	173·08	5·10	..	..	..	..	..	..	..	..	..	5·879	6·421

TABLE IIIA.—TABLE RECORDING THE FLOW OF BRINE, CORRECTED TO EQUIVALENT FLOW AT 10° C. THROUGH BATH WEATHERSTONE 4½ INCHES THICK.

Number of Experiment.	Density of Solution.			Difference in Density, Inlet and Outlet. *	Pressure. Lbs. Square Inch.	Discharge in Gallons per Hour per Square Foot at 10° C.		Difference in Flow of Fresh Water and Brine.	Temperature of Inlet Solution °C.
	Fully Saturated at 15° C.	Inlet.	Outlet.			Fresh Water.	Brine.		
1	1·2043	1·182	1·018	+0·164	20	0·125	0·174	+0·049	13·0
2	”	1·182	1·018	+0·164	28	0·203	0·129	−0·074	13·0
3	”	1·182	1·064	+0·118	30	0·232	0·121	−0·111	13·0
4	”	1·186	1·155	+0·031	37	0·321	0·186	−0·135	15·2
5	”	1·160	1·175	−0·015	37	0·321	0·180	−0·141	15·2
6	”	1·175	1·156	+0·019	37	0·321	0·208	−0·113	15·2
7	”	1·155	1·162	−0·007	40	0·366	0·204	−0·162	15·5
8	”	1·155	1·160	−0·005	40	0·366	0·276	−0·090	15·5
9	”	1·006	1·135	−0·129	32	0·257	0·155	−0·102	30·0
10	”	1·006	1·146	−0·140	36	0·309	0·171	−0·138	30·0
11	”	1·035	1·101	−0·066	28	0·203			

* NOTE. + signifies that the inflowing brine is of greater density than the percolating discharge.
− ” ” ” ” ” ” ” less ” ” ” ” ”

TABLE IV.—TABLE SHOWING THE VARIATION OF FLOW OF WATER WITH PRESSURE THROUGH VARIOUS ROCKS.

Flow expressed in Gallons per Hour, per Square Foot of Exposed Surface, per Lb. of Pressure per Square Inch, on that Surface.

Surface Pressure in			Chalk.		Bath Weatherstone.		Bradford Oolite.		Daresbury Sandstone.			Millstone Grit.	
			Mottisfont.	Micheldever.					Hard.		Soft.		
Lbs. per Square Inch.	Feet of Water.	Atmospheres.	Thickness in Inches 6	Thickness in Inches 6	Thickness in Inches 4·5	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 3	Thickness in Inches 6	Thickness in Inches 6	Thickness in Inches 3
5	11·59	0·34	0·0494	0·0554	0·0036	0·0018	0·0020	0·0050	0·6000	1·0213	1·4200	0·0158	0·1206
10	23·08	0·68	0·0430	0·0564	0·0036	0·0018	0·0020	0·0049	0·7300	1·2620	1·8120	0·0197	0·1235
15	34·61	1·02	0·0409	0·0553	0·0047	0·0019	0·0021	0·0048	0·9333	1·7807	1·9826	0·0246	0·1239
20	46·15	1·36	0·0397	0·0558	0·0062	0·0020	0·0025	0·0048	0·9875	1·6321	2·0105	0·0283	0·1235
25	57·69	1·70	0·0404	0·0536	0·0072	0·0022	0·0028	0·0048	0·9192	1·6028	2·0076	0·0312	0·1237
30	69·23	2·04	0·0423	0·0532	0·0077	0·0024	0·0031	0·0051	0·8750	1·6043	2·0263	0·0341	0·1235
35	80·77	2·38	0·0473	0·0529	0·0084	0·0026	0·0033	0·0056	0·8340	1·5683	2·0428	0·0378	0·1239
40	92·31	2·72	..	0·0521	0·0091	0·0028	0·0036	0·0061	0·8025	1·5005	2·0437	0·0417	0·1226
45	103·85	3·06	..	..	0·0095	0·0031	0·0038	0·0075	0·7758	1·4587	..	0·0455	0·1218
50	115·38	3·40	..	..	0·0105	0·0033	0·0040	0·0078	0·7506	1·4032	..	0·0523	0·1183
55	126·92	3·74	..	..	0·0117	0·0035	0·0042	0·0089	0·7295	1·3491	..	0·0651	0·1111
60	138·46	4·08	..	..	..	0·0037	0·0044	0·0101	0·7042	1·3023	..	0·0839	0·1033
65	150·00	4·42	..	..	..	..	0·0047	0·0117	0·6784	1·2431	..	0·0835	0·0962
70	161·54	4·76	..	..	..	..	0·0049	..	0·6524	1·1794	..	0·0812	0·0905
75	173·08	5·10	..	..	..	..	..	..	..	..	..	0·0784	0·0856

TABLE V.—TABLE SHOWING THE POROSITY, ETC., OF VARIOUS STONES.

Rock.	Thickness of Test-piece in Feet.	Mean Diameter of Constituent Particles in Inches.	Unit Porosity of Rock.	Net Volume of Test-piece in Cubic Feet.	Total Volume of Pores in Test-piece in Cubic Feet.	Discharge Factor for a Discharge of 1 Gallon per Square Foot of Rock Surface.	Discharge Factor multiplied by Discharge per Square Foot per Hour at 5 Lbs. Pressure.	Discharge Factor by Discharge per Hour at 5 Lbs. Pressure Reduced to Multiples of the Least.
Mottisfont Chalk . . .	0·5	0·0088	0·4586	0·393	0·180	0·699	0·1727	10·28
Micheldever ,, ...	0·5	0·0058	0·4679	0·393	0·184	0·684	0·1895	11·28
Bath Weatherstone . . .	0·375	0·0110	0·2418	0·294	0·071	1·773	0·0319	1·90
,, ,, . . .	0·25	0·0110	0·2418	0·196	0·047	2·679	0·0241	1·43
Bradford Oolite	0·5	0·0087	0·1902	0·393	0·075	1·678	0·0168	1·00
,, ,, ...	0·25	0·0087	0·1902	0·196	0·037	3·403	0·0851	5·06
Hard Daresbury sandstone .	0·5	0·0120	0·2519	0·393	0·099	1·272	3·8160	227·15
,, ,, ,, . .	0·25	0·0120	0·2519	0·196	0·049	2·569	13·1276	781·40
Soft ,, ,, . .	0·5	0·0119	0·2315	0·393	0·091	1·383	9·8193	584·48
Millstone Grit	0·5	0·0289	0·1498	0·393	0·059	2·134	0·1686	10·03
,, ,, ...	0·25	0·0289	0·1498	0·196	0·029	4·341	2·6176	155·81

TABLE VI.—TABLE SHOWING THE REDUCED PRESSURES CORRESPONDING TO VARIOUS SURFACE PRESSURES WITHIN VARIOUS ROCKS.

Rock.	Thickness of Rock in Inches.	Surface Pressures in Lbs. per Square Inch.													
		5	10	15	20	25	30	35	40	45	50	55	60	65	70
Reduced Pressures in Lbs. per Square Inch at a Depth of 1 Inch below Surface.															
Bath Weatherstone . .	3	4·0	8·3	12·4	16·8	21·0	25·1	29·3	33·7	37·8	42·1	46·3	50·5	54·7	58·9
Bradford Oolite . . .	6	3·6	7·3	11·1	14·9	18·8	22·4	26·2	30·0	33·8	37·5	41·3	45·2	49·0	52·8
,, ,, . . .	3	3·9	7·9	11·9	15·9	19·8	23·8	27·6	31·7	35·6	39·6	43·6	47·5	51·4	55·2
Hard Daresbury sandstone	6	3·0	6·0	9·0	12·0	15·1	18·2	21·2	24·1	27·2	30·2	33·2	36·1	39·1	42·1
,, ,, ,,	3	3·5	7·1	10·7	14·3	17·9	21·5	25·1	28·6	32·2	35·8	39·3	42·9	46·5	50·2
Soft ,, ,,	6	4·2	8·4	12·8	17·0	21·1	25·3	29·6	34·0	38·2	42·4				
Millstone Grit	3	3·9	7·9	11·8	15·7	19·5	23·3	27·3	31·2	35·1	39·0	42·5	46·8	50·6	54·5
Reduced Pressures in Lbs. per Square Inch at a Depth of 2 Inches below Surface.															
Bath Weatherstone . .	3	3·1	6·4	9·9	13·0	16·3	19·8	23·0	26·3	29·7	33·0	36·2	39·7	43·0	46·2
Bradford Oolite . . .	3	2·2	4·4	6·5	8·7	10·8	13·0	15·2	17·4	19·6	21·7	23·9	26·1	28·3	30·4
Hard Daresbury sandstone	3	2·9	5·8	8·7	11·6	14·5	17·4	20·2	23·1	26·1	29·0	31·9	34·9	37·8	40·7
Millstone Grit	3	3·1	6·3	9·4	12·5	15·7	18·9	22·1	25·3	28·4	31·6	34·8	38·1	41·2	44·3
Reduced Pressures in Lbs. per Square Inch at a Depth of 3 Inches below Surface.															
Micheldever Chalk . .	6	4·2	8·4	12·6	16·8	21·1	25·3	30·0	33·6						
Bradford Oolite . . .	6	2·5	5·4	8·3	11·1	14·0	16·9	19·8	22·6	25·3	28·2	31·1	34·0	36·9	39·7
Hard Daresbury sandstone	6	2·1	4·1	6·2	8·2	10·4	12·5	14·5	16·4	18·6	20·9	23·0	25·1	27·2	29·4
Soft ,, ,,	6	3·1	6·7	10·0	13·3	16·8	20·0	23·3	26·9	30·1	33·7				
Reduced Pressures in Lbs. per Square Inch at a Depth of 5 Inches below Surface.															
Micheldever Chalk . .	6	3·5	7·0	10·6	14·2	17·8	21·2	25·0	28·3						
Bradford Oolite . . .	6	1·7	3·6	5·4	7·4	9·3	11·2	13·1	15·0	17·0	18·8	20·7	22·7	24·5	26·3
Hard Daresbury sandstone	6	0·7	1·4	2·1	2·7	3·4	4·1	4·8	5·5	6·2	6·8	7·5	8·2	9·0	9·7
Soft ,, ,,	6	1·9	3·9	5·8	7·8	9·6	11·4	13·3	15·2	17·2	19·1				

TABLE VII.—TABLE SHOWING THE LOSS OF PRESSURE PER LB. OF SURFACE PRESSURE AT VARIOUS DEPTHS FROM THE SURFACE.

Rock.	Thickness of Test-Piece.	Per Cent. Pore Space.	Size of Grains. Diameter.	Loss of Pressure per Lb. of Pressure at Inches below Surface.					Loss of Pressure per Lb. of Pressure per Inch of Depth from Surface at Inches below Surface.				
				1	2	3	4	5	1	2	3	4	5
	Inches.		Inch.										
Micheldever Chalk . .	6	46·79	0·0058	0·060	0·114	0·159	0·208	0·295	0·060	0·057	0·053	0·052	0·059
Bath Weatherstone . .	3	24·18	0·0110	0·160	0·340	..	..	..	0·160	0·170	..	..	..
Bradford Oolite . . .	6	19·02	0·0087	0·250	0·340	0·438	0·532	0·625	0·250	0·170	0·146	0·133	0·125
" " . . .	3	..	..	0·208	0·564	..	..	..	0·208	0·282	..	..	..
Hard Daresbury sandstone	6	25·19	0·0120	0·396	0·498	0·582	0·704	0·865	0·396	0·249	0·194	0·176	0·173
" " "	3	..	..	0·284	0·418	..	..	..	0·284	0·209	..	..	..
Soft " "	6	23·15	0·0119	0·152	0·236	0·327	0·456	0·615	0·152	0·118	0·109	0·114	0·123
Millstone Grit	3	14·98	0·0289	0·220	0·366	..	..	..	0·220	0·183	..	..	..

TABLE VIII.—TABLE SHOWING THE QUANTITY OF WATER ABSORBED BY A PIECE OF UPPER CHALK FROM MICHELDEVER, AFTER REPEATED IMMERSIONS, EACH OCCUPYING FROM 1 TO 2 SECONDS.

Dimensions, 9·70×3·89×3·21 centimetres. Volume, 121·12 cubic centimetres. Specific gravity, 1·545. Percentage pore space, 46·79.

Number of Immersions.	Quantity of Water Absorbed.			Temperature °C.			Surface Dry After Immersion.	Water absorbed expressed as an Enveloping Film. Centimetre Thick.
	In Grams.	As Per Cent. of Volume of Stone.	As Per Cent. of Pore Space.	Air.	Water.	Chalk.		
1	15·85	13·09	27·97	16	15	83	instantly	0·0974
2	24·35	20·11	42·96			Not recorded	,,	0·0523
3	28·35	23·41	50·01				,,	0·0243
4	32·45	26·80	57·28				,,	0·0254
5	36·85	30·42	64·99				,,	0·0270
6	39·35	32·49	69·44				,,	0·0154
7	40·25	33·23	71·02				,,	0·0056
8	41·55	34·30	73·31				,,	0·0079
9	42·35	34·96	74·73				5 seconds	0·0050
10	43·05	35·55	75·99				30 ,,	0·0043
11	43·45	35·87	76·68				40 ,,	0·0024
12	43·55	35·96	76·85	16	15	33	50 ,,	0·0007

TABLE VIIIA.—TABLE SHOWING THE RELATIVE ABSORPTION OF TWO PIECES OF UPPER CHALK.

Treatment.	Volume of Stone in Cubic Centimetres.		Total Time of Immersion in Hours.	Temperature °C.	Absorption as a Percentage of Volume of Stone.	
	Micheldever.	Mottisfont.			Micheldever.	Mottisfont.
Immersion in an open pan with 1 foot of water above stone . .	73·75	44·07	0·5	10	39·76	36·03
			4·5		40·28	36·78
			47·0		42·38	42·87
Immersion in a shallow pan with ⅜ inch water above stone, under the receiver of an air-pump.			1·5	10	41·49	36·58
Total pore space . .					46·79	45·86

TABLE VIIIB.—TABLE SHOWING THE RATE OF DRYING A PIECE OF MOTTISFONT CHALK.

Treatment.	Temperature °C.	Hours from the Commencement of Drying.	Water Evaporated Expressed as a Percentage of the Volume of the Stone.
Dried in air	9·0	23·5	6·96
,, ,, ,, ...	10·1	48·0	8·02
,, ,, ,, ...	8·1	95·5	10·58
,, ,, ,, ...	8·4	119·0	12·00
,, ,, ,, ...	8·7	166·0	18·66
,, ,, steam oven . .	96·0	172·0	45·80

TABLE VIIIc.—TABLE SHOWING SOME PROPERTIES OF FILTER SAND.

No. of Strands per lineal Inch.	Diameter of Wire in Inches.	Net Area of Opening in Square Inches.	Specific Gravity of the Sand.	Weight of Sifted Sand in Grammes.	Volume as a Percentage of the Total Volume.	Adherent Water per unit volume of Sand.	Adherent Water in gallons per cubic foot.
3·6	0·025	0·064	1·611	2·47	0·09	..	..
6	0·015	0·023	1·633	102·12	3·78	0·1582	0·99
10	0·010	0·0081	1·672	10·67	0·38	0·2660	1·66
20	0·010	0·0016	1·622	373·50	13·90	0·5501	3·43
30	0·010	0·00053	1·603	457·08	17·22	0·4963	3·10
60	0·005	0·00014	1·606	1,208·82	45·44	0·4848	3·03
90	0·005	0·00004	1·619	434·36	16·20	0·4642	2·90
through 90	..	..	1·634	80·99	2·99	0·4748	2·96

TABLE VIIId.—RECORD OF A SERIES OF EXPERIMENTS WITH SOME OF THE ABOVE SAND.

	Meshes per Linear Inch.			
	20	30	60	90
Volume of sand in cubic centimetres "dry loose"	265	340	270	325
Volume of sand in cubic centimetres "dry shaken"	235	300	240	285
Reduction in volume as a percentage of the volume "dry shaken" . .	12·77	13·33	12·50	14·04
Volume of water in cubic centimetres added to sand	230	213	215	290
Volume of water and sand presuming no pores	465	513	455	575
Volume of water and sand actual measurement	365	405	355	472
Difference of theoretical and actual volume of sand and water, i.e., volume of contained water as a percentage of volume of sand "dry shaken"	42·55	36·00	41·66	36·14
Volume of sand in water "shaken wet"	245	305	245	287
Increase of volume "shaken wet" as a percentage of volume "shaken dry"	4·25	1·66	2·08	0·70
Difference of theoretical and actual volume corrected for the increased volume "shaken wet," i.e., true volume of contained water . .	40·81	35·41	40·81	35·89
Adherent water in loose condition calculated on the volume of dry compact sand	55·01	49·63	48·48	46·42
Difference of percentages of contained water under the two conditions .	14·20	14·22	7·67	10·53

TABLE IX.—TABLE SHOWING DETAILS OF

Authority Owning Works.	Geological Formation.	Pumping-Station at
Penzance Corporation	Granite elvans	Polteggan
Barnoldswick U. D. C.	Millstone grit	Whitemoor
Barry U. D. C.	Dolomitic conglomerate	Biglis
" "	" "	"
Hucknall Torkard U. D. C.	Permian sandstone	Calverton No. 1
" " "	" "	" " 2
Birmingham Corporation	Triassic sandstone	Aston Well, No. 1
" "	" "	" " " 2
" "	" "	King's Vale Well " " " bore
" "	" "	Short Heath Well
" "	" "	Selly Oak "
" "	" "	Long Bridge "
" "	" "	Perry Sinking
" "	" "	" Well
Gloucester Corporation	" "	Newent
Hinckley U. D. C.	" "	Snarestone
Pickering G. & W. Co.	" "	Fieldhead
Runcorn U. D. C.	" "	Runcorn
St. Helen's Corporation	" "	Eccleston Hill
" " "	" "	Whiston
" " "	" "	Knowsley
" " "	" "	Kirkby
Stafford Corporation	" "	Milford
Stourbridge W. W. Co.	" "	Mill Meadow
" "	" "	Coalbournbrook
" "	" "	Tack
Wallasey U. D. C.	" "	Poulton No. 1
" "	" "	" " extended
" "	" "	" " with air lift
" "	" "	Poulton No. 2

[1] Rest level of water is at surface.
[2] Well would overflow, if pumping ceased.
[3] Working level at surface.
[4] Where italic figures are used the

BRITISH WATERWORKS PUMPING-STATIONS.

Depth in Feet Below Surface of				When Pumping		Diameter of Well in Feet						
Ordnance Datum.	Bottom of Well or Bore.	Rest Level of Water.	Working Level of Water.	Gallons per Day.	Hours per Day.	At Surface	At Depth from Surface: In Feet.	At Depth from Surface: Dia.	At Depth from Surface: In Feet.	At Depth from Surface: Dia.	At Depth from Surface: In Feet.	At Depth from Surface: Dia.
220·0	..	12·0	12·6	200,000	*24·0*[6]	..	..	..	..	..	..	..
..	600	22·0	100·0+[4]	120,000	8·0	6·66	100	6·66	600	1·0	..	..
21·87 21·87	32 40	[1] at surf.	5·0 5·0	650,000	24·0	7·5 11·0	32 40	7·5 11·0	.. ..	.. ..	.. ..	.. ..
276·0 276·0	125 125	80·0 80·0	96·0 96·0	14,000	15·0	8·0 10·0	125 125	8·0 10·0	.. ..	.. ..	.. ..	.. ..
302·60	127	11·0	109·0	660,384	24·0	10·0	127	10·0	..	..	..	..
305·68	353	14·0	109·0	1,288,077	24·0	10·0	28	10·0	135	1·5	..	..
451·56	172 400	29·0	162·0	213,400	7·0	13·0 1·66	172 400	13·0 1·66	..	..	..	..
354·84	300	−4·0[2]	84·0	1,790,000	24·0	20·0	36	20·0	44	17·0	104	9·5
450·06	300	98·0	114·0	629,321	16·0	12·5	150	12·5	300	1·5	..	..
562·36	496	[1] at surf.	105·0	323,800	24·0	10·0	201	10·0	496	0·33	..	..
326·80	1,488	20·0	200·0	600,000	24·0	16·0	395	16·0	1488	0·50	..	..
326·40	168	14·8	95·0	1,425,600	24·0	13·0	168	13·0	..	..	..	..
100·0	170	3·0	50·0	600,000	10·0	..	..	..	..	..	..	..
255·0	..	65·0	145·0	180,000	*10·0*[6]	..	..	..	..	..	..	..
..	..	2·0	2·0	50,000	*10·0*[6]	..	..	..	..	..	..	..
219·0	361	219·0	262·0	400,000	20·0	26 × 8[7]	271	30 × 10[7]	361	..	..	..
260·0	388	140·0	200·0	550,000	24·0	7·0	210	7·0	210	0·83	388	0·83
200·0	465	198·0	212·0	450,000	24·0	9·0	225	9·0	225	1·5	465	1·5
140·0	699	90·0	165·0	1,000,000	24·0	11·0	173	11·0	173	2·0	699	1·5
100·0	510	30·0	120·0	1,500,000	24·0	11·0	150	11·0	150	2·0	510	1·5
247·38	239	12·69	23·1	600,000	14·66	10·0	42	9·5	42	0·5	239	0·5
237·0	130	− x[3]	25·0	500,000	24·0	6·0	50	6·0	50	1·25	130	1·25
218·0	177	− x[3]	at surf.[5]	750,000	24·0	..	64	..	64	..	177	..
222·0	220	20·0	30·0	300,000	24·0	5·0	20	5·0	20	..	220	..
38·0	400	81·0	102·0	425,000	24·0	7·0	72	7·0	72	1·08	400	1·08
38·0	700	81·0	102·0	160,000	24·0	as above		..	400	0·84	700	0·84
38·0	700	81·0	102·0	450,000	24·0	as above		..	..	..	..	..
38·0	400	81·0	94·0	550,000	24·0	7·0	81	7·0	81	1·25	400	1·25

[3] Rest level of water is at 358·84 O.D. 4·00 feet above floor level.

[4] Working level of water in driest season.

values are assumed.

[7] Well is elliptical in cross section.

TABLE IX.—

Authority Owning Works.	Geological Formation.	Pumping-Station at
Wallasey U. D. C.	Triassic sandstone . .	Liscard . . . . . . ; ,, bore outside .
,, ,, ...	,, ,, . .	,, extended two bores in well . .
,, ,, ...	,, ,, . .	Liscard reserve, bore No. 1; ,, ,, ,, ,, 2
Warrington Corporation . . .	,, ,, . .	Winwick . . . . .
,, ,, . . .	,, ,, . .	Myddleton . . . . .
Scarborough ,, . . .	Oolite	Osgodby . . . . .
,, ,, . . .	,, ...	Irton . . . . . .
Filey U. D. C.	,, ...	Filey . . . . . .
Ryde Corporation	Lower greensand . .	Knighton . . . . .
Bournemouth G. & W. Co. . .	Chalk	Wimborne . . . . .
Chepping Wycombe Corporation .	,, ...	High Wycombe . .
Devizes Corporation	,, ...	Shepherd's Shore . .
Dover Corporation	,, ...	Connaught Road . .
East Surrey W. W. Co. . . .	,, ...	Kenley . . . . . .
,, ,, ,, . . .	,, ...	Purley . . . . . .
Faversham W. W. Co. . . .	,, ...	Copton, Ospringe . .
Frimley and Farnborough W. W. Co.	,, ...	Itchell, Crondall . .
Henley-on-Thames W. W. Co. . .	,, ...	Henley . . . . . .
Kingston-upon-Hull Corporation .	,, ...	Cottingham . . . .
,, ,, ,, ,, .	,, ...	Springhead . . . .
Lewes W. W. Co.	,, ...	Southover . . . .
Marlborough Corporation . . .	,, ...	Marlborough . . .
Ryde Corporation	,, ...	Ashey . . . . . .
Saffron Walden Corporation . .	,, ...	Saffron Walden . .
Southampton ,, . .	,, ...	Otterbourne . . . .
Windsor Corporation	,, ...	Tangier Island . . .
Arbroath Burgh Council . . .	Gravel	Nolt Loan . . . . .
Tonbridge W. W. Co.	,, ...	Tonbridge . . . . .

[1] Rest level of water 10 feet above surface.
[2] Maximum depression.

continued.

Depth in Feet Below Surface of				When Pumping		Diameter of Well in Feet						
Ordnance Datum.	Bottom of Well or Bore.	Rest Level of Water.	Working Level of Water.	Gallons per Day.	Hours per Day.	At Surface	At Depth from Surface: In Feet.	Dia.	At Depth from Surface: In Feet.	Dia.	At Depth from Surface: In Feet.	Dia.
109·0 109·0	166 600	130·0	158·0	1,250,000	24·0	12·0 1·5	166 600	.. 1·25	.. ..	.. ..	.. ..	.. ..
109·0	300 800	130·0	175·0	2,250,000	24·0	12·0 as above	166 300	12·0 3·5	166 600	3·5 2·0	300 800	3·5 1·25
100·0	800 900	130·0	180·0	2,250,000	24·0	2·75 2·75	300 300	2·75 2·75	600 600	2·0 2·0	800 900	1·25 1·25
110·0	..	110·0	140·0	1,400,000	12·0	9·0	..	..	..	..	..	..
62·0	..	50·0	80·0	1,300,000	12·0	12·5	..	..	..	..	..	..
161·77	100	68·8	79·8	864,000	24·0	10·0	100	10·0	..	..	..	..
96·0	428	−10·0[1]	6·0	1,250,000	24·0	10·0	70	10·0	98	2·0	250 428	1·66 1·0
120·0	110	60·0	72·0	80,000	13·0	6·0	110	6·0	..	..	..	..
..	108	10·0	50·0	400,000	24·0	7·5	66	7·5	108	1·5	..	..
66·0	256	at surf.[2]	100·0	2,000,000	24·0	11·0	60	11·0	97 167	10·0 9·0	210 256	10·0 0·5
225·0	154	8·0	12·0	230,000	*24·0*[4]	14·0	..	..	..	..	..	..
561·0	138	74·0	107·0	160,000	10·0	..	140	..[5]	140 140	1·0 2·0	190 190	1·0 2·0
211·0	200	194·0	215·0	1,250,000	*24·0*[4]	6·0	..	..	..	..	..	..
180·0 240·0	.. ..	30·0 60·0	50·0 80·0	2,500,000	*24·0*[4]	..	..	..	..	..	..	..
230·0	..	90·0	130·0	480,000	*24·0*[4]	..	..	..	..	..	..	..
275·0	300	at surf.[2]	48·0	200,000	24·0	8·0	50	8·0	50	..	300	..
121·0	..	25·0	27·0	215,000	*24·0*[4]	0·66	.. ?	..	..	..	..	..
30·28	86	10·28	34·0	5,000,000	15·5	13·75	86	13·75	..	..	..	..
16·09	73	8·59	29·0	5,900,000	20·0	22·0	32	21·5	73	21·5	..	..
22·0	120	5·0	16·0	500,000	10·0	6·0	22	6·0	120	1·25	..	..
499·9	143	94·8	..	60,000	10·0	..	..	..	..	..	..	..
78·0	80	25·0	75·0	200,000	24·0	7·5	..	..	..	..	..	..
155·4	350	4·0 4·0	23·0 23·0	12,000 360,000	10·0 24·0	1·21	82	1·21	82	0·71	350	0·71
90·0	100	17·5	50·8[3]	4,000,000	24·0	6·0	100	6·0	..	..	..	..
19·34	135	20·0	24·0	1,000,000	24·0	..	..	..	..	..	..	..
57·0	..	37·0	53·0	280,000	*24·0*	..	..	..	..	..	..	..
74·0	23	12·0	16·0	360,000	*24·0*	..	..	..	..	..	..	..

[2] Rest level of water at surface.
[4] Where *italic* figures are used, the values are assumed.
[5] Two wells 20 feet apart connected by a gallery 5 feet in width.

TABLE X.—TABLE SHOWING THE

NOTE.—*A + sign after the figures in column 5 indicates overflow.*
rest-level, column 9, should be increased by an amount
at Wallasey are not now in use, and the Liscard

District.	Pumping-Station at	Geological Formation.	Yield of Well in Cubic Feet per Second.	Height in Feet above Bottom of Well or Bore of Water at		Drop in Feet of Water-Level when Pumping
				Rest Level.	Working Level.	
Barnoldswick . .	Whitemoor . .	Millstone grit .	0·668	578	500	78
Barry	Biglis . . .	Dolomitic conglomerate .	1·205	40	35	5
		,, ,,	..	32	27	5
Hucknall Torkard	Calverton No. 1.	Permian sandstone	0·042	45	29	16
	,, No. 2.	,, ,,	..	45	29	16
Birmingham . .	Aston No. 1. .	Triassic ,,	1·232	116	18	98
	Aston No. 2. .	,, ,,	2·404	339	244	95
	King's Vale . .	,, ,,	1·366	371	238	133
	,, ,, . .	,, ,,	..	143	10	133
	Short Heath .	,, ,,	3·341	304	216	88
	Selly Oak . .	,, ,,	1·762	202	186	16
	Longbridge . .	,, ,,	0·604	496	391	105
	Perry Sinking .	,, ,,	1·130	1,468	1,288	180
	Perry Well . .	,, ,,	2·661	153	73	80
Gloucester . .	Newent . . .	,, ,,	2·688	167	120	47
Hinckley . . .	Snarestone . .	,, ,,	0·334	..	..	80
Pickering . . .	Fieldhead . .	,, ,,	0·224	..	..	0
Runcorn . . .	Runcorn . . .	,, ,,	0·896	133	99	43
St. Helen's . .	Eccleston Hill .	,, ,,	1·051	248	188	60
	Whiston . . .	,, ,,	0·839	267	253	14
	Knowsley . .	,, ,,	1·867	609	534	75
	Kirkby . . .	,, ,,	2·799	480	390	90
Stafford . . .	Milford . . .	,, ,,	1·833	226	216	10
Stourbridge . .	Mill Meadow .	,, ,,	0·933	130+	105	25+
	Coalbournbrook .	,, ,,	1·400	177+	177	? +
	Tack	,, ,,	0·560	200	190	10
Wallasey . . .	Poulton No. 1 .	,, ,,	0·793	319	298	21
	,, ,, .	,, ,,	0·297	619	598	21
	,, .	,, ,,	0·840	619	598	21
	Poulton No. 2 .	,, ,,	1·030	357	344	13
	Liscard Well .	,, ,,	2·332	36	8	28
	Liscard Bore. .	,, ,,	..	470	442	28

YIELD, ETC., OF VARIOUS WELLS.

Where adits are mentioned in the Remarks column, the volume at
equal to the capacity of the adit. The Poulton wells
wells are not worked simultaneously, but alternately.

Depth in Feet pene-trated into Water-bearing Rock.	Volume of Water in Cubic Feet in Well and Bore at		Difference in Cubic Feet of Volume at Rest and at Working Levels.	Highest Rest Level.		Lowest Rest Level.		Maximum recorded Variation of Rest Level in Feet.	Remarks.
	Rest Level.	Working Level.		Height above O.D.	Date.	Height above O.D.	Date.		
..	2,718	393	2,325						
28	3,802	3,327	475	22·12	Dec. 1899	19·87	July, Aug. Sept. several years	2·25	
20	1,414	1,193	1,221						
125	3,534	2,278	1,256	196·00	Dec. 1904	190·00	Jan. 1900	6·00	Adit connecting wells.
125	2,262	1,458	804						
..	9,190	1,414	7,776	..	..	..	..	..	Adit connecting the two wells.
..	9,890	2,427	7,463						
..	618	396	222	..	..	..	..	..	Adit connecting bore and well.
..	18,975	1,327	17,548						
..	18,906	2,951	15,955						
..	6,647	4,683	1,964	..	..	..	..	..	Adits.
..	15,812	7,566	8,246	..	..	..	..	..	Adits.
..	75,585	39,405	36,180	..	..	..	..	..	Adit.
..	20,335	9,686	10,649						
140									
388	2,791	482	2,309	172·00	Aug. 1858	120·00	Dec. 1904	52·00	
..	2,142	1,251	891	30·00	Apr. 1875	2·00	Dec. 1904	28·00	
..	9,152	2,025	7,127	55·00	Aug. 1890	50·00	Mar. 1905	5·00	
510	12,270	3,717	8,553	80·00	Oct. 1890	70·00	Nov. 1903	10·00	
200	2,227	1,450	777	237·14	Apr. 1900	233·56	July 1905	3·58	
130	1,512	805	707						
177	..	..	..	218·00	..	187·00	Mar. 1905	31·00	
220	638	442	196	220·00	..	210·00	Mar. 1905	10·00	
360	294	275	19						
660	457	438	19	..	..	..	..	..	Extension of bore.
660	457	438	19	..	..	..	..	..	Extension, with air lift.
360	392	376	16						
146	4,068	904	3,164						
580	656	608	48	..	..	..	..	..	Connected to well outside.

TABLE X.—

District.	Pumping-Station at	Geological Formation.	Yield of Well in Cubic Feet per Second.	Height in Feet above Bottom of Well or Bore of Water at		Drop in Feet of Water-Level when Pumping
				Rest Level.	Working Level.	
Wallasey . . .	Liscard Well, bore, etc. .	Trias sandstone . .	..	170	125	45
	,, ,,	,, ,, . .	4·199	670	625	45
	Liscard reserve: Bore No. 1 .	,, ,, . .	4·197	670	620	50
	Liscard reserve: Bore No. 2 .	,, ,, . .	..	770	720	50
Warrington . .	Winwick . .	,, ,, . .	..	38	8	30
	,, ,,	,, ,, . .	5·227	38	8	30
	,, ,,	,, ,, . .	..	38	8	30
	Myddleton . .	,, ,, . .	4·851	122	92	30
Scarborough . .	Osgodby . .	Oolite, coralline . .	1·613	31	20	11
	Irton . . .	,, ,, . .	2·333	428+	422	16
Filey	Filey . . .	,, calcareous grit	0·276	15	3	12
Ryde	Knighton . .	Cretaceous, lower greensand . . .	0·748	98	58	40
Bournemouth .	Wimborne . .	Cretaceous, upper chalk	3·733	256	61	195
Chepping Wycombe	High Wycombe	,, ,, ,,	0·429	146	142	4
Dover	Connaught Road	,, ,, ,,	2·318	..	..	21
East Surrey . .	Kenley . . . / Purley . . .	,, ,, ,, / ,, ,, ,,	4·635	..	..	20
Faversham . .	Copton, Ospringe	,, ,, ,,	0·890	..	..	40
Frimley . . .	Itchell, Crondall	,, ,, ,,	0·373	300	252	48
Henley . . .	Henley . . .	,, ,, ,,	0·399	..	..	2
Hull	Cottingham .	,, middle ,,	1·445	75+	51	24
	Springhead . .	,, ,, ,,	10·975	64+	42	22
Lewes	Southover . .	,, upper ,,	2·240	115	104	11
Ryde	Ashey . . .	,, ,, ,,	0·372	55	5	50
Saffron Walden .	Saffron Walden	,, ,, ,,	0·537	346	327	19
Southampton . .	Otterbourne .	,, ,, ,,	7·418	82	49	33
Windsor . . .	Tangier Island .	,, ,, ,,	1·855	115	111	4
Arbroath . . .	Nolt Loan . .	Gravel	0·519	..	..	16
Tonbridge . . .	Tonbridge . .	,, ...	0·668	11	7	4

continued.

Depth in Feet penetrated into Water-bearing Rock	Volume of Water in Cubic Feet in Well and Bore at		Difference in Cubic Feet of Volume at Rest and at Working Levels.	Highest Rest Level.		Lowest Rest Level.		Maximum recorded Variation of Rest Level in Feet.	Remarks.
	Rest Level.	Working Level.		Height above O.D.	Date.	Height above O.D.	Date.		
280	3,323	1,202	121	..	..	..	..	..	Bore in bottom of well.
780	4,510	2,389	2,121	..	..	..	..	..	,, ,,
780	2,197	1,900	297	..	..	..	..	..	No. 1 bore } in use alternately with the Liscard well.
880	2,320	2,023	297	..	..	..	..	..	No. 2 bore }
..	1,908	509	1,399						
..	1,908	509	1,399	..	..	..	..	..	Adit 5 × 3·5 × 3,900 feet.
..	1,908	509	1,399						
..	14,973	11,290	2,683	..	..	..	..	..	Adit 6 × 4 × 3,600 feet; rise 1 in 1,200 from well.
60	2,454	1,588	866	100·0	Mar. each year	93·0	Oct. each year	7·00	
300	6,852	6,381	471	106·0					
..	424	85	339	60·0	Apr. May each year	47·5	Aug. Sept. each year	12·50	
..	2,548	781	1,767	..	Dec. each year				
113	16,448	10,275	6,173						
..	..	..	..	..	..	..	..	..	Three wells 6 feet diameter 200 feet deep, and adit 6 × 4 × 600 feet.
..	2,513	101	2,412						
40	11,095	7,570	3,525						
40	23,782	15,438	8,344						
..	601	290	311	19·0	Feb. 1904	15·5	Dec. 1901	3·50	
80	2,429	221	2,208	200·0	Feb. each year	150·0	Sept. each year	50·00	
..	195	173	22	155·4	Feb. 1904	150·9	Mar. 1903	4·50	
80	4,665	2,782	1,882	..	..	..	..	..	2 wells and adits.
7	..	..	..	..	..	..	..	..	Two tubes 90 feet apart, and heading 500 yards long.

TABLE XI.—TABLE SHOWING DETAILS O

Authority.	Filter Beds at	No. of Beds.	Area in Square Yards. Total.	Area in Square Yards. Average in Use.	Gallons Treated per Day.
Barrow-in-Furness Corporation	Peake Beck . . .	4	5,760	*5,760*	3,900,000
Birmingham Corporation . .	Whitacre, Plants Brook and Aston . . .	12	39,944	26,984	13,666,000
Bishop Auckland U. D. C. .	West Mills	2	1,200	*1,200*	380,000
Bolton Corporation	Heaton, Sweetloves and Belmont	9	15,258	10,890	6,250,000
Bradford „ (Yorks.) .	Gilstead	6	22,000	18,333	7,000,000
„ „	Thornton Moor . .	4	6,667	5,000	2,500,000
„ „	Chellow Heights . .	5	12,500	10,000	5,000,000
Burnley „	Cant Clough . . .	3	4,450	*2,970*	1,031,000
Cheltenham „	Tewkesbury . . .	4	1,320	*1,320*	700,000
„ „	Dowdeswell . . .	7	2,155	*2,155*	1,000,000
Chester Water Works Co. .	Riverside	6	7,812	6,510	1,705,200
Derby Corporation	Little Eaton . . .	9	5,350	4,218	2,600,000
Doncaster „	..	4	1,568	*1,568*	994,000
Drogheda „	..	3	16,200	*10,800*	200,000
Dumfries W. W. Commissioners	Lochfoot	6	3,060	*2,550*	1,173,000
Exeter Corporation . . .	Pynes	6	8,028	6,690	1,750,000
Falkirk Water Trust . . .	..	6	2,300	*1,917*	2,200,000
Harrogate Corporation . . .	Harlow Hill and Iron-gate Reservoirs . .	11	6,190	4,000	1,700,000
Hay, Brecknock, U. D. C. .	..	1	450	*450*	35,000
Higham Ferrers and Rushden Water Board	Sywell	3	2,016	*1,344*	550,000
Jersey New W. W. Co. . .	..	5	2,180	*1,744*	250,000
Keighley Corporation . . .	Oldfield, Oakworth .	4	5,200	3,900	1,300,000
Kilmarnock Burgh Council .	..	9	5,100	*4,534*	1,644,000
Knutsford G. & W. Co. . .	..	2	511	*511*	129,000
Leicester Corporation . . .	..	21	20,366	*17,460*	5,091,000
Lincoln „ . . .	..	6	4,396	*3,664*	*1,480,000*
Loughborough „ . . .	Nanpanton	5	1,435	1,122	700,000
Malvern U. D. C.	British Camp Reservoir	3	915	610	220,000
Nelson Corporation	..	2	1,250	*1,250*	982,000
Paignton U. D. C.	Blagdon	4	364	*273*	200,000
Paisley Burgh Council . . .	..	17	10,200	*8,400*	4,553,000
Ripon Corporation	Lumley Moor . . .	2	800	400	200,000 to 400,000
Rothesay Burgh Council . .	..		1,164	*776*	90,000
Rugby U. D. C.	Avon Water Works .	3	1,500	*1,000*	680,000
Tees Valley Water Board . .	Broken Scar, Darlington	11	15,677	13,252	7,500,000
Tiverton Corporation . . .	..	3	380	*254*	300,000
Todmorden „ . . .	Gorpley	5	1,590	640	200,000
Tunbridge Wells Corporation .	..	4	4,840	*3,630*	712,000
Ulverston U. D. C. . . .	Castle Hill	2	1,500	750	400,000
Wakefield Corporation . . .	Kirkhamgate . . .	5	10,000	8,000	2,500,000
Warrington „	Appleton	3	1,370	*914*	400,000 to 600,000
Weardale and Consett W. Co.	Waskerley Reservoir .	5	4,632	3,706	1,667,000
„ „ „ „ .	Honey Hill	5	5,385	4,308	2,000,000
„ „ „ „ .	Tunstall Reservoir . .	4	5,866	4,400	2,000,000
Worcester Corporation . . .	..	9	6,401	*5,690*	1,507,000
York W. W. Co.	Acomb Landing . .	8	15,100	15,100 ?	3,500,000

NOTE.—Where italic figures ar

BRITISH FILTER BEDS (SAND AND GRAVEL).

Thickness of the Filtering Material of Undernoted Approximate Sizes.														Bed Cleaned Every —Days.
Fine Sand.	1/16 Inch.	1/8 Inch.	3/16 Inch.	3/8 Inch.	1/2 Inch.	3/4 Inch.	1 Inch.	1¼ Inch.	2 Inch.	2½ Inch.	3 Inch.	4 Inch.	6 Inch.	
Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	Ft. In.	
1 8				1 0				1 0		1 8				190
3 0				0 7½			0 7½		1 3					24
3 0		1 0					1 0		1 0			1 0		42
2 6		0 6												7–21
3 0		0 6			0 6			0 6			0 6		0 9	10–12
3 6		0 6			0 6			0 6			0 6		2 0	10–12
2 6	0 6				and tiles laid flat on drain									10–12
2 0	0 6				0 9					0 9		0 9		42
3 6	1 0													14
3 6	1 0													21
4 2														42
	2 6		0 6	0 6		0 6		0 6			1 0			14
	0 6				3 0								0 9	42
2 0			0 3					1 6					0 9	14
1 6					0 6							0 6	1 6	28
3 6		0 3			0 6									21
	1 6					1 0		2 0						14
				also three gravity filters 8 feet in diameter										
0 6	0 9	0 9							1 0					30
0 9										0 9			1 0	18
					also Reeve's system									
3 0		0 5	0 5				1 6							
2 0		2 0												
2 0							1 6			2 6				14
3 1					0 7									
1 0	1 0	0 6	0 6		0 6						1 6			31
3 0		0 4		0 4		0 4								21–42
	2 0	1 0												365
2 0		0 6				0 6			0 6				1 0	28
2 0		0 6				0 6			0 6				1 0	28
1 6		0 6				0 6			0 6				1 0	28
4 0		2 6												

inserted the values are assumed.

TABLE XII.—TABLE SHOWING THE DISCHARGE, ETC., OF BRITISH FILTER-BEDS.

Filter Beds at	Discharge per Square Foot of Bed per Hour. In Gallons.	Discharge per Square Foot of Bed per Hour. In Cubic Feet.	Thickness in Feet of the Fine Sand.	Thickness in Feet of the Coarse and Fine Sand and Fine Gravel.	Thickness in Feet of the Whole Bed.	Height in Feet of Water above the Top of Bed.	Total Head in Feet.	Average Area of each Bed in Square Yards.
Barrow in Furness	3·16	0·502	1·67	2·67	5·33	1·5	6·83	1,440
Birmingham . .	2·34	0·375	3·00	3·62	5·50	2·0	7·50	3,329
Bishop Auckland	1·48	0·237	3·00	4·00	7·00	2·0	9·00	600
Bolton . . .	2·66	0·426	2·50	3·00	3·00	1·0	4·00	1,695
Bradford (Yorks)	1·79	0·287	3·00	4·00	5·75	3·5	9·25	3,667
,, ,,	2·30	0·369	3·50	4·50	7·50	3·5	11·00	1,666
,, ,,	2·31	0·370	2·50	3·00	3·16	3·5	6·66	2,500
Burnley . . .	1·60	0·256	2·00	3·25	4·75	1·5	6·25	1,483
Cheltenham . .	2·45	0·393	3·50	4·50	4·50	1·5	6·00	330
,, . .	2·15	0·344	3·50	4·50	4·50	1·5	6·00	308
Chester . . .	1·21	0·194	4·16	..	4·16	3·3	7·49	1,302
Derby . . .	2·85	0·456	2·50	3·50	5·50	2·0	7·50	594
Doncaster . .	2·93	0·469	..	..	..	..	..	392
Drogheda . .	0·09	0·014	..	..	..	..	..	5,400
Dumfries . .	2·13	0·341	..	..	..	..	..	510
Exeter . . .	1·21	0·194	0·50	3·50	4·25	2·5	6·75	1,338
Falkirk . . .	5·31	0·852	..	..	..	..	..	383
Harrogate . .	1·97	0·316	2·00	2·25	4·50	0·75	5·25	563
Hay	0·36	0·058	..	..	..	..	..	450
Higham Ferrers .	1·89	0·304	..	..	..	..	..	672
Jersey . . .	0·66	0·106	..	..	..	..	..	436
Keighley . . .	1·54	0·247	1·50	2·00	4·00	1·42	5·42	1,300
Kilmarnock . .	1·68	0·269	..	..	..	..	..	567
Knutsford . .	1·17	0·187	..	..	..	..	..	255
Leicester . .	1·35	0·216	..	..	..	..	..	970
Lincoln . . .	1·87	0·300	..	..	..	..	..	733
Loughborough .	2·26	0·362	3·50	4·25	4·25	2·50	6·75	287
Malvern . . .	2·47	0·397	1·50	1·50	4·50	1·00	5·50	305
Nelson . . .	3·64	0·582	..	..	..	..	..	625
Paignton . . .	2·84	0·407	0·50	2·00	3·00	11·50	14·50	91
Paisley . . .	2·51	0·402	..	..	..	..	..	600
Ripon . . .	2·31 4·63	0·370 0·742	0·75	0·75	2·50	2·00	4·50	400
Rothesay . .	0·54	0·086						
Rugby . . .	2·10	0·337	3·00	3·84	5·33	2·50	7·83	500
Tees Valley . .	2·44	0·391	2·00	4·00	4·00	2·00	6·00	1,425
Tiverton . . .	3·66	0·587	2·00	2·00	6·00	5·50	11·50	127
Todmorden . .	1·45	0·232	3·08	3·66	3·66	3·00	6·66	318
Tunbridge Wells	0·91	0·146	..	..	..	..	..	1,210
Ulverston . .	2·47	0·396	1·00	3·50	5·00	2·00	7·00	750
Wakefield . .	1·45	0·232	3·00	3·66	4·00	2·50	6·50	2,000
Warrington . .	1·35 2·03	0·216 0·325	2·00	3·00	3·00	2·00	5·00	457
Weardale and Consett . .	2·08	0·333	2·00	2·50	4·50	1·75	6·25	926
,, ,,	2·15	0·344	2·00	2·50	4·50	1·75	6·25	1,077
,, ,,	2·10	0·337	1·50	2·00	4·50	1·75	6·25	1,466
Worcester . .	1·23	0·196	..	..	..	..	..	711
York	1·07	0·171	4·00	6·50	6·50	2·50	9·00	1,887

See note on previous Table of details.

OBITUARY.

ALEXANDER BEAZELEY, who died on the 1st December, 1905, was the eldest son of the late Commander George Beazeley, R.N. Born at Brighton, on the 6th August, 1830, he was articled in 1846 to the late Mr. John Wright, and while yet a pupil was employed by him in setting out the Portsmouth Direct Railway. In 1852 Mr. Beazeley went out to New South Wales, and became an Assistant, and afterwards Executive Engineer in the Public Works Department of the Colony, in which capacity he supervised the construction of lighthouses at Sydney, Cape Howe, and Gabo Island, Moreton Bay, and of various bridges and other works. In 1863 he returned home, and subsequently acted as Resident Engineer on the Kidwelly Branch of the Cardigan and Carmarthen Railway. In 1865 he entered the service of the Trinity House as assistant to the Chief Engineer, and had charge of the construction of various lighthouses round the coast. He was an authority on fog-signals, and was the author of a Paper[1] on the subject read before this Institution in 1871, for which he was awarded a Telford medal and premium. In 1873 he went to Sweden as Resident Engineer on the Halmstad and Jönköping line and afterwards on the North of Europe Railway. On his return from Sweden he became Librarian to the Royal Institute of British Architects, an appointment which he held for many years. He was the Author of "Tables of Tangential Angles and Multiples for setting out Curves," and of "The Reclamation of Land from Tidal Waters." He edited and practically rewrote Usill's "Practical Surveying" and was also one of the earliest and most constant contributors to the Oxford English Dictionary, having furnished over 30,000 words to that work. He was an accomplished linguist in French, German, Italian and Swedish, the last named of which he spoke and wrote with exceptional fluency and ease.

Mr. Beazeley was elected a Member of the Institution on the 4th December, 1860.

[1] Minutes of Proceedings Inst. C.E., vol. xxxii, p. 95.

WILLIAM DEAN, formerly Locomotive, Carriage and Wagon Superintendent of the Great Western Railway, died after a long illness at Folkestone on the 24th September, 1905, aged 65. The second son of the late Mr. Henry Dean, the subject of this notice was born in London on the 9th January, 1840, and received his education at the Haberdashers' Company's School. On leaving school in 1855 he became a pupil to the late Mr. Joseph Armstrong in the locomotive shops of the Great Western Railway at Wolverhampton. Having completed his articles, he became one of the engineering staff and in 1864 he was appointed assistant superintendent of the locomotive department under Mr. George Armstrong. The duties of this position included the management of the works at Stafford Road, Wolverhampton, where 850 men were employed and 20 new engines built annually, in addition to ordinary repairs and renewals. In 1868 he was transferred to the locomotive, carriage and wagon department at Swindon as chief assistant superintendent, and on Mr. Joseph Armstrong's death in 1877, Mr. Dean was appointed Locomotive, Carriage and Wagon Superintendent of the Great Western Railway, a position which he held until his retirement, owing to ill-health, in 1902.

During his occupation of the post of Superintendent, the department underwent considerable expansion and development, the number of men employed when he retired being about 26,000. Many improvements in the design and construction of locomotives and other rolling stock were introduced under Mr. Dean's able direction, and not a few changes which have added to the comfort and convenience of passengers were the outcome of his skill and ingenuity. He was greatly respected by his colleagues and the men, and his retirement and the failing health to which it was due caused widespread regret.

Mr. Dean was keenly interested in the Volunteers, and early in his career he joined the 2nd V.B. Duke of Edinburgh's Wiltshire Regiment. For many years at Swindon he worked with steady enthusiasm in connection with the movement, and at the time of his death he held a major's commission in his old regiment. He was also a Justice of the Peace for Wiltshire. He was a Member of the Institution of Mechanical Engineers, and served on the council of that society from 1892 until his death.

Mr. Dean was elected an Associate Member of this Institution on the 6th March, 1877, and was transferred to the class of Members on the 15th January, 1878.

GEORGE FOOT, born on the 12th June, 1833, received his scientific training at Trinity College, Dublin, where he graduated in Arts and obtained the diploma in Engineering in 1855. After serving for several years under the late Mr. W. R. Le Fanu on the construction of branches of the Great Southern and Western Railway of Ireland, he came to London and was appointed by Mr. Charles Blacker Vignoles, Past-President, an assistant engineer on the construction of the Tudela and Bilbao Railway in Spain. Leaving Spain in 1864, he went out to Mexico in the following year as Resident Engineer on the construction of the Mexican Railway, becoming subsequently Chief Resident Engineer, and, in 1872, Engineer-in-Chief of the line. He also held the appointment of Consulting Engineer, during construction, of the Mexican Central Railway between Mexico City and Leon. He died in Mexico City on the 14th January, 1906, aged 72.

Mr. Foot was elected a Member of the Institution on the 7th December, 1880.

GEORGE LEEDHAM FULLER died at Theale, Somerset, on the 2nd January, 1906, at the advanced age of 83. In early years, after being regularly trained for the engineering profession, he was employed as locomotive superintendent on the Bristol and Gloucester line and as Assistant Resident Engineer on the construction of the Wilts, Somerset and Weymouth Railway. In 1852 he engaged in practice on his own account and was consulted principally in connection with slate-quarries in North Wales. He acted as Consulting Engineer to several companies, and also acquired and worked a slate-quarry himself, but was eventually obliged to give this up owing to lack of sufficient capital. In the eighties he prepared plans for a breakwater and sea-wall at Criccieth, and superintended the erection of the latter. In 1856 he took out a patent for an improved compound engine, a model of which is in the South Kensington Museum. He retired in 1895 from the active pursuit of his profession, but continued to take a keen interest in the proceedings of the Institution, with which he was connected for nearly 50 years.

Mr. Fuller was elected an Associate of the Institution on the 7th December, 1858, and was transferred to the class of Members on the 6th December, 1864.

JOHN HOWKINS was born in September, 1839, at Granton, where his father was acting as Resident Engineer on the harbour-works then under construction for the Duke of Buccleuch. The subject of this notice was educated in Edinburgh, and served a pupilage of 5 years to the late Mr. James Leslie, for whom he subsequently superintended the repair of a breach in the West Pier of Methill Harbour in Fifeshire. After spending a short time in the office of Messrs. M'Clean and Stileman, in Westminster, he was in 1862 appointed Resident Engineer by the Harbour Trustees of Stranraer to superintend the completion of a low-water steamboat-pier designed by Mr. Leslie for the accommodation of steamers engaged in the Irish trade. On the completion of this work in 1863, he returned to Messrs. M'Clean and Stileman's office, and at the end of that year was appointed Resident Engineer of the new bridge at Weybridge designed by that firm. The bridge was opened in 1865, when Mr. Howkins was transferred to the Furness and Midland Railway, and was engaged in surveying and setting out the section of that line between Wennington and Carnforth. In the following year, he succeeded Mr. Frank M'Clean as Resident Engineer on the Devonshire Dock works, then under construction by his firm at Barrow-in-Furness, and superintended these works until after the opening of the dock in 1867.

Mr. Howkins commenced private practice in Edinburgh in 1868, but having obtained the appointment of Engineer to the Hartlepool Port and Harbour Commissioners in 1870, he removed to Hartlepool, and for the next 7 years was employed in carrying out the improvements which the Commissioners had begun in 1869 from the designs of Mr. Thomas Dyke, who was then their Engineer. The principal undertakings were the extension of the breakwater and the removal of the bar at the entrance to the outer harbour of Hartlepool: the successful accomplishment of the latter led to the construction by the North Eastern Railway Company of the large docks and timber-ponds which now occupy the site of the Old Slake.

Mr. Howkins left Hartlepool in 1877 to become Engineer and General Superintendent of Granton Harbour, in succession to his father, a position which he retained until his death. He designed and carried out all the works connected with that harbour undertaken during his tenure of office, and was occasionally consulted in regard to similar works elsewhere, notably the improvement of the harbour of Scarborough carried out between the years 1878 and 1881. For many years his services were retained as Consulting Engineer to the Commissioners of the Port and Harbour

of Hartlepool. Mr. Howkins died at Granton on the 26th April, 1906, aged 66.

He was elected an Associate of the Institution on the 5th February, 1867, and was transferred to the class of Members on the 3rd December, 1872.

GEORGE LEWIS, born at Coleorton, Leicestershire, on the 7th April, 1836, was trained under Mr. J. C. Bailey, of Wigan, in the mining branch of the profession, to which his career was subsequently devoted. Between 1861 and 1866 he was Manager of the Coleorton and Swannington Collieries, Leicestershire, where he sank two shafts, put down winding-plant, erected the necessary machinery and constructed sidings and branch lines. Shortly after the latter date, Mr. Lewis engaged in independent practice as a mining engineer, and in course of time, became associated with most of the principal collieries and coal-fields of the Midlands, as well as with gypsum and ironstone mines in that district. He was retained as Engineer to the Donisthorpe, Coleorton, Wollaton, Manners, South Leicestershire, Ibstock, Whitewick and other collieries, representing an aggregate annual output of nearly a million tons. From Coleorton he transferred his headquarters to Derby, where he continued to practise until his death, which took place at his residence, Green Hill, Derby, on the 9th November, 1905. Mr. Lewis was a Fellow of the Geological Society and a Member of the Institution of Mining Engineers, of which he was President in 1893. He was also for some years a member of council of the Mining Association of Great Britain.

He was elected a Member of this Institution on the 3rd February, 1885.

WILLIAM McLANDSBOROUGH, born on the 14th April, 1829, at Otley, Yorkshire, was trained for the engineering profession under his father, the late Mr. John McLandsborough, and subsequently acted as Resident Engineer on various water-supply, drainage and sewerage works carried out under the direction of Sir Robert Rawlinson, Past-President, and other prominent engineers. Between 1861 and 1864 he occupied the position of Engineer to the City of Gloucester, and in the latter year he was appointed Engineer and Manager of the Cheltenham Waterworks, subsequently becoming Waterworks Engineer to the borough on the transfer of the water

undertaking to the Corporation. During his tenure of the latter office, the waterworks were largely developed under his supervision. The works at Tewkesbury, designed by the late Mr. Thomas Hawksley, Past-President, were constructed by him, as well as the large reservoir at Dowdeswell, and he also superintended the construction of Evesham waterworks, and smaller works at Winchcombe. In 1890 Mr. McLandsborough retired from his appointment under the Corporation, after a connection with the waterworks extending over 26 years of able and devoted service, the value of which was fully appreciated by his employers and by the community. After his retirement he was frequently consulted upon water-supply schemes and similar undertakings, in connection with which he was associated with several prominent engineers and gave evidence before parliamentary committees and at arbitrations. He died suddenly at his residence in Cheltenham on the 5th December, 1905, in his seventy-seventh year.

Mr. McLandsborough was elected an Associate of the Institution on the 24th May, 1859, and was transferred to the class of Members on the 30th April, 1878.

DAVID JAMES ROSS, born in London on the 5th February, 1845, was educated at various public schools and served his articles between 1861 and 1864 as a pupil to Mr. John Foster Pickering, Surveyor, of Old Jewry. After the expiration of his pupilage he gained further experience in the offices of several well-known surveyors and engineers, and in November, 1871, he entered the engineer's department of the Commissioners of Sewers, under the late Colonel Haywood. After passing through all the departments, he was appointed to the position of Chief Assistant in 1882, and on the death of Colonel Haywood in 1894, he succeeded that gentleman in the appointment of Engineer to the Commissioners of Sewers. In 1897 the Commission was amalgamated with the Corporation of London, Mr. Ross being retained as Engineer to the Public Health Department. During his official career he was responsible for the laying out of a complete system of fire-hydrants, the construction of extensive sewerage-works in connection with the various Metropolitan and other railway extensions, the electric lighting of the City streets, the formation of new streets and subways, and generally the works appertaining to the lighting, paving and control of the public thoroughfares. The

street-works comprised the widening of Lower Thames Street, the improvement at the west end of Cheapside, the formation of Lloyd's Avenue, with subway and sewer, and the widening of Fleet Street, Fenchurch Street, and Gracechurch Street. He also erected the City shelter in Golden Lane, the Crematorium at Ilford Cemetery, and several underground conveniences in the City. Mr. Ross retired in June, 1905, in consequence of ill-health, and in October, after a long illness, he underwent a serious operation; this in itself was successful, but his strength being exhausted, he failed to recover, and died on the 29th October, 1905, aged 60.

He was a Member of the Surveyors' Institution and of the Incorporated Association of Municipal and County Engineers.

Mr. Ross was elected an Associate Member of this Institution on the 5th December, 1882, and was transferred to the class of Members on the 15th January, 1895.

CARL HEINRICH VON SIEMENS[1] died at Mentone on the 21st March, 1906, aged 77. Born in 1829 at Menzendorf in Mecklenburg, he was the sixth of eleven sons in a family which has given illustrious names to engineering science. The greater part of his career is indissolubly bound up with those of his three brothers, William (afterwards Sir William),[2] Werner[3] and Friedrich, whilst with the two first-named he helped to carry on the various electrical undertakings with which the name of Siemens is identified. As an inventor his name is not so well known as those of his brothers, but as a capable business man and organizer, distinguished for untiring energy and perseverance, his career has left its mark on all the undertakings with which he was associated.

His father, Christian Ferdinand Siemens, died in 1839, and after residing for a time in Lübeck, the family removed to Berlin, where the subject of this notice completed his education. Later on he became the assistant, in the firm of Siemens and Halske, of his eldest brother, the late Dr. Werner von Siemens, whom he helped to lay the first underground telegraph-wires. In 1848, William, Carl and Friedrich followed their elder brother to the war, the last two entering the Schleswig-Holstein army as volunteers,

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[1] This notice is based on that which appeared in *Engineering*, 30th March, 1906.

[2] Minutes of Proceedings Inst. C.E., vol. lxxvii, p. 352.

[3] *Ibid*, vol. cxii, p. 339.

where they remained till the conclusion of the armistice. Carl then entered a chemical factory in Berlin, which he soon quitted to assist Werner in the reconstruction of the telegraph-lines. In 1851 he and Friedrich represented the Berlin firm at the London Universal Exhibition, and afterwards Carl was appointed manager of the Paris branch of the firm, but in 1852 he came over to London to take charge of the London office for his brother William, who was then engaged at Birmingham on his work in connection with the regenerative steam-engine. In the same year the Berlin firm was entrusted with important telegraph work by the Russian Government, and this resulted in the appointment of Carl to take complete charge of their interests in Russia, where mainly as a result of his energy and ability the operations of the firms rapidly attained considerable proportions. In 1855, the year in which he took charge of a large branch factory at St. Petersburg, Mr. Carl Siemens married the daughter of Mr. Kapherr, the firm's previous representative in that city. In 1864 Mr. Halske severed his connection with the London firm, which was reconstituted under the style of Siemens Brothers and was carried on by Dr. Werner Siemens, of Berlin, Mr. (afterwards Sir William) Siemens, of London, and Mr. Carl Siemens, of St. Petersburg, as partners. Five years later Mr. William Siemens, finding that his private work took up a great part of his time, asked his brother at St. Petersburg to undertake the management of the London business. During this period Carl had erected all the more important telegraph-lines in Russia under great difficulties, but the expiration of the Russian maintenance-contracts having somewhat curtailed his activity in that country, he was able to consent to his brother's wishes. About this time, moreover, he had the misfortune to lose his wife, who died in St. Petersburg. Coming to England in 1869, he managed the London business in conjunction with his brother William for 11 years, the Woolwich works being gradually enlarged and a gutta-percha factory established. Amongst other work he took an active part in the laying of submarine cables, taking charge in 1874 of the expedition of the firm's new cable-ship "Faraday" on her maiden voyage, for the purpose of laying the Direct Atlantic Cable, which was the first transatlantic cable made by the firm of Siemens Brothers. He was also the first to succeed in recovering from the depths of the Atlantic a broken submarine cable. In 1880 the firm of Siemens Brothers was converted into a limited liability company under its present title, Mr. Carl Siemens becoming one of the directors; but the dampness of the English climate affecting his health, he was obliged to return to Russia with his children. He settled down in St. Petersburg, where

his health improved, and again undertook the management of the Russian branch of the business, which under his able control soon recovered its former prominence. His two daughters married in Russia and his son assisted him in the business. This branch of the firm played a leading part in the development of the electric-lighting and power industry in Russia, and has increased rapidly in size, now giving employment to nearly a thousand hands. When Dr. Werner von Siemens died in 1892, Carl Siemens was called to Berlin to take the leadership of the firm of Siemens and Halske. In 1899 he was elected chairman of Siemens Brothers and Company, Limited, which position he retained until his death. In 1895 the Russian hereditary rank of nobility was conferred upon him. He was a Member of the Institution of Electrical Engineers from its foundation.

The relations of the four brothers throughout their lives were of the most intimate and affectionate kind. It is therefore fitting to close this memoir by quoting from Dr. Werner von Siemens's "Personal Recollections" the tribute which he paid to his brother's character: "I should call the next brother Charles the most normally constituted of us all. He was always to be depended upon, faithful and conscientious, a good pupil, an affectionate, attached brother. His clear eye and generally cultivated understanding made him an excellent man of business, and, with his large technical knowledge and an excellent tact, an admirable conductor of business undertakings. Charles was the true connecting link between us four brothers, who differed indeed radically from one another, but were bound together for lifelong common work by all-subduing fraternal love."

Carl Heinrich von Siemens was elected a Member of this Institution on the 24th May, 1870.

JAMES EDWARD TUIT, born on the 2nd January, 1860, was educated at King's College, London, and received his practical training under the late Mr. Edmund Olander, in the engineering department of the Great Western Railway, where he developed marked ability in constructional design. In 1882 he joined the staff of the late Sir John Fowler and Sir Benjamin Baker, Past-Presidents, who were then engaged on the designs of the Forth Bridge. For 2 years Mr. Tuit was occupied on the details of these designs and in 1884, when the work of construction commenced, he was appointed by the contractors an assistant engineer at Queensferry, and was responsible for carrying out the temporary works required in the erection of the bridge. He remained at the works until their completion in 1889,

when he came to London to take sole charge of the erection of the Tower Bridge for Sir William Arrol and Company, the contractors. During the construction of this great work, Mr. Tuit found full scope for the exercise of his talent and ingenuity in dealing with the numerous problems which arose in the course of its erection. Upon the completion of the Tower Bridge, Mr. Tuit became responsible for the designing work for Sir William Arrol and Company, and took charge of their London office. Bringing to his task exceptional constructional ability and a keen mathematical mind, the designs which he evolved were of sound practical value, combining strength and lightness of proportion with considerable grace of form.

The last important design upon which he was engaged, but which he was not destined to see completed, was that for a bridge across the Nile at Cairo, with two smaller bridges connecting the mainland with an island in the river. The Egyptian Government invited competitive designs, and that submitted by Mr. Tuit was accepted, not only because it harmonized, from an æsthetic point of view, with the architectural and scenic features of the surroundings, but because it met the severe limitations involved, and gave assurance that the difficulties in regard to foundations had been carefully considered and would be successfully overcome. Mr. Tuit was subject to severe dysentery and had been for some time in indifferent health. Latterly his condition had improved, but whilst at his office he suddenly became seriously ill, and being removed to his home, he died the next day, the 20th February, 1906, at the comparatively early age of 46.

He was elected an Associate Member of the Institution on the 1st December, 1885, and was transferred to the class of Members on the 23rd January, 1894.

CHARLES BEWICKE BLACKBURN, born on the 22nd May, 1875, was educated at Hulme Grammar School and the Owens College, Manchester, and graduated in science at the Victoria University. He obtained his practical training in the works of the Electric Construction Company, Limited, and remained in their service on the termination of his pupilage, first as assistant to his father, Mr. A. B. Blackburn, and afterwards as Assistant Manager of the works at Wolverhampton, employing about 800 men and manufacturing all classes of electrical machinery. In 1902 he resigned this position, and after serving for a short time in the Brush Electrical Company's works at Loughborough, he went out to the United States and

obtained employment with the General Electric Company at Schenectady for about a year. He relinquished this employment in order to take up the pursuit of agriculture at Hemet in California, where, 18 months afterwards, on the 26th June, 1905, he met his death whilst shooting, as the result of the accidental discharge of his gun.

Mr. Blackburn was elected an Associate Member of the Institution on the 3rd December, 1901.

JOHN BOWDEN, born on the 8th May, 1841, near Morpeth, Northumberland, became assistant surveyor to the Preston Corporation in 1863, and after serving in this capacity for a period of 5 years, obtained an appointment as District Surveyor to the Salford Corporation. His connection with the latter body lasted until 1872, during which time he was engaged in the design and execution of extensive sewerage and other works, including the main intercepting sewer of the borough to Mode Wheel.

In 1872 Mr. Bowden was appointed Surveyor of the Trafford Estates near Manchester and in Cheshire, a position which he held for a period of 31 years. During this period he also engaged in private practice as an architect and surveyor in Manchester, and carried out several important drainage schemes in the neighbourhood. He died at Glossop, Derbyshire, on the 16th November, 1905, in his sixty-fifth year.

Mr. Bowden was elected an Associate of the Institution on the 2nd February, 1869, and was subsequently placed in the class of Associate Members.

JOHN CARLINE, second son of the late Mr. Thomas Carline, for many years County Surveyor to the North Riding of Yorkshire, was born in 1839 and received his education at the Grammar School, Richmond, Yorkshire. After serving 3 years in his father's office, he joined the staff of the late Sir Thomas Bouch, by whom he was employed on the construction of the South Durham and Lancashire Union Railway, connecting the North Eastern and the London and North Western systems. This line traverses very high ground and its construction presented considerable engineering difficulties. Mr. Carline superintended the erection of some of the highest viaducts on this railway, and subsequently acted as Assistant Resident

Engineer on the extension of the line to West Auckland. In 1862 he was appointed Resident Engineer in charge of the construction of the Leadburn, Linton and Dolphinton branch of the North British Railway, successfully carrying the line over Leadburn Moss, one of the deepest mosses in the south of Scotland, and in the following year he held sole charge of the works of the Edinburgh, Perth and Dumfriesshire Railway, comprising several heavy bridges over the water of Leith and extensive sewer- and road-diversions. In 1869 Mr. Carline received the appointment of Highways Surveyor and subsequently that of Surveyor to the Lewisham District Board of Works, an appointment which he held until 1901, when the office of Surveyor to the Board of Works was abolished, a period of more than 30 years. As Surveyor, he designed and constructed all the sewers, roads, bridges and other improvement works carried out by the Board in the extensive and steadily growing suburban district under their care, involving a considerable annual expenditure. Mr. Carline died at Melrose, Scotland, on the 26th September, 1905, aged 66. He was a Member of the Sanitary Institute and of the Incorporated Association of Municipal and County Engineers.

He was elected an Associate Member of this Institution on the 4th December, 1877.

WILLIAM RICHARD HOPKINS CHIPPERFIELD, Borough Engineer of Ladysmith, Natal, died on the 13th March, 1906, from injuries sustained in a fall from his horse. Born on the 22nd June, 1857, he served his pupilage to Mr. Baldwin Latham, with whom, on the completion of his articles, he remained as an assistant, afterwards becoming Chief Assistant and Resident Engineer on numerous works of water-supply, sewerage and sewage-disposal for which Mr. Latham was the Engineer. After 14 years' service with that gentleman, he obtained the appointment of Engineer and surveyor to the Atherstone district, Warwickshire, for which, under Mr. Latham's direction, he had carried out water-supply and sewerage works. He retained this position until 1902, a period of 12 years, when he went out to South Africa and joined the Public Works Department of Cape Colony. He obtained the post of Chief Assistant Engineer at Port Elizabeth, and during the time he was thus employed, he prepared two schemes on biological principles for dealing with the sewerage of places of 10,000 inhabitants, and was also engaged in connection with drainage and water-supply schemes at Grahamstown and Middelburg. In September,

1904, Mr. Chipperfield received the appointment of Borough Surveyor of Ladysmith, with which duties were combined those of waterworks engineer to the town. He at once set to work energetically upon his new duties, and was engaged in filtration experiments and other improvements of the water-supply when he met with the accident which led to his untimely death, deeply regretted by the community to whom he had given undoubted proofs of his ability and devotion.

Mr. Chipperfield was elected an Associate Member of the Institution on the 29th May, 1883.

ARTHUR HILL GODFREY, born on the 14th March, 1863, practised for several years, at the beginning of his career, as a Government land surveyor in the Cape Colony and Bechuanaland. In 1888 and the following year, he served a pupilage to Mr. R. E. Brounger, who at that time, under an arrangement between the Cape Colony and the Orange Free State, was building the railway from the Orange River to the Vaal River. Mr. Godfrey did excellent work when superintending the construction of the line between Bloemfontein and the Vaal River, a distance of 213 miles.

On the completion of this work, he entered into partnership with Mr. Baynon at Johannesburg, practising as civil engineers and surveyors and carrying out surveys of land, mines and railways, the last under contract with the Netherlands South African Railway Company. When war broke out, Mr. Godfrey left Johannesburg and again obtained employment on the Cape Government Railways as Engineer in charge of surveys between Eende Kuil and Clanwilliam, and Butterworth and Umtata, 180 miles in all. The underground work entailed in the course of the numerous mine-surveys which he conducted, however, had seriously affected his lungs, and in 1905, owing to failing health, he was obliged to leave the railway service. He died at Cradock, Cape Colony, on the 13th February, 1906.

Mr. Godfrey was elected an Associate Member of the Institution on the 3rd March, 1891.

JOHN NEWMAN, third son of the late Mr. Orlando Newman, of Paddington, was born on the 3rd March, 1847, and after being privately educated, received his practical training under the late

Mr. James McNair Harkness, of Westminster. In 1867 he obtained employment on the Limehouse Basin improvement works of the Regents Canal Company, and on the completion of these works he became contractor's engineer in sole charge of the construction of the Kington and Eardesley railway, from the commencement until its completion in 1873, after which he carried out in the same capacity the Kington and Radnor extension of the same railway. Subsequently he was employed on the construction of the Ballymena and Portglenone railway and in other railway work.

Mr. Newman was the Author of several works on engineering subjects, including "Notes on Concrete," "Earthwork Slips and Subsidences," "Metallic Structures, Corrosion, Fouling, etc.," and "Cylinder Bridge Piers," copies of which are in the Library of the Institution. He also contributed frequently to the columns of the technical press and occasionally lectured before the students of the Royal Engineers' Institute, Chatham. He died on the 11th February, 1906, in his fifty-ninth year.

Mr. Newman was elected an Associate Member of the Institution on the 12th May, 1874.

ALEXANDER BROWN PORTUS, born on the 21st February, 1834, at Black Creek, Hunter River, New South Wales, received his training as an engineer and millwright under his father, Mr. John Portus, with whom, after undertaking a voyage to Europe and America in 1855, to make himself acquainted with engineering practice in those countries, he was engaged in the design and erection of various classes of machinery until 1865. In the latter year, he entered the Public Works Department of New South Wales, and after carrying out dredging work on the Hunter River and at Newcastle, he was appointed in 1880 Superintending Engineer of all dredging-plant in New South Wales, at first under the late Mr. E. O. Moriarty, and subsequently under Mr. C. W. Darley. He retained this appointment until his retirement, owing to failing health, in 1904. During his occupation of the position of Superintendent, considerable additions were made to the comparatively small fleet of dredging-machines which the State possessed in 1880, and much valuable land was reclaimed and other improvements effected. On his initiative centrifugal-pump dredging was introduced into the Colony, and the work done was described by Mr. Portus in Papers read before the Royal Society of New South Wales. Eighteen powerful suction dredgers were also added, and have greatly reduced

the cost of removing sand. The twin-screw hopper dredgers built to his design, drawing only 5½ feet of water and capable of steaming and dredging at the same time, proved an unqualified success in the rough water on the sandy bars of the rivers on the coast of New South Wales. Many improvements in connection with dredging were devised by Mr. Portus, and the gimbal joints and rings for connecting pipes on pontoons, extensively used in the Colony, were introduced by him. His skill and ability were recognized not only by the Government of New South Wales, but also by other States and private bodies who availed themselves of his professional services. He died at his residence, Moore Park, Sydney, on the 8th October, 1905, in his seventy-second year.

Mr. Portus was elected an Associate Member of the Institution on the 7th February, 1888.

HAMLET ROBERTS, born on the 3rd June, 1853, served his articles in the engineering department of the Pearson Knowles Coal and Iron Company, Warrington, at the expiration of which, in 1874, he obtained employment on the staff of the Gas and Water Engineer of Birkenhead. After 6 years' experience at Birkenhead, he secured the appointment of Superintendent of the Dover water-supply, a position which he held until 1891, when he resigned to take charge of the erection of the Colonial Sanitary Engineering Works at Dronfield. On the completion of these works, Mr. Roberts was appointed Waterworks Engineer of Ipswich, a post which he retained until his death. The water undertaking had recently been purchased by the Corporation from the Ipswich Waterworks Company and needed improvement in various directions, work to which Mr. Roberts devoted his best energies. The works and machinery having become inadequate to meet the growing needs of the town, considerable extensions became necessary and were carried out under his active supervision. These included the erection of a new pumping-plant, duplicated several years later, for which he designed and built large engine- and boiler-houses, the extension of the mains throughout the borough, and the design and construction of an additional service reservoir, having a capacity of 4½ million gallons. These works were completed and the water-supply brought into efficient working order, but whilst engaged in testing mains which were to be laid to give winter work to the unemployed, Mr. Roberts, who never enjoyed

robust health, caught a severe chill. This developed into pleuro-pneumonia, from which he died on the 12th October, 1905, aged 52.

He was elected an Associate Member of the Institution on the 2nd March, 1897.

JOSEPH HENRY WOOLCOCK, born on the 20th January, 1867, was the son of the late Mr. Henry Woolcock,[1] mining engineer, under whom he served his pupilage between 1883 and 1887. In 1891, he was taken into partnership by his father, the firm practising under the style of Henry Woolcock and Son. On his father's death, which took place in the following year, Mr. Woolcock became solely responsible for the extensive consulting practice of the firm, and had charge as Engineer and Manager of mines belonging to Fletcher's High House Mining Company, Falcon Mining Company, Asby Colliery Company and the Dalmellington Iron Company. For the last-named company he superintended the sinking of a new shaft and designed pumping-engines and other machinery. He also acted as Mining Engineer in the West Cumberland district to the London and North Western, and Furness Railways, and as consulting engineer to several mining lessors. He was a Member of the North of England Institute of Mining and Mechanical Engineers. At the time of his death, he was chairman of the St. Bees Gas Company, and for some years had done good work as a member of various local bodies. In 1904, being then in failing health, he took his chief assistant, Mr. W. W. Smith, into partnership, the title of the firm being changed to Woolcock and Smith. Mr. Woolcock died suddenly at St. Bees on the 30th December, 1905, aged 38.

He was elected an Associate Member of this Institution on the 7th February, 1893.

JOHN MACKAY, who died at Hereford on the 5th February, 1906, was one of the last of that band of men who, in conjunction with the late Mr. Thomas Brassey, took an active part in the construction of railways in this country almost from their commencement. Born at Bogart, Sutherland, on the 25th October, 1822, the subject of this notice entered Mr. Brassey's service in 1843, and was employed on the construction of railways at first in France and Belgium, and subsequently in England on the Shrewsbury and Hereford, the Shrewsbury

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[1] Minutes of Proceedings Inst. C.E., vol. cx, p. 390.

and Crewe, and many other lines in Shropshire, North Staffordshire, and Central and South Wales. On the death of Mr. Brassey in 1870, Mr. Mackay commenced contracting on his own account, and carried out the Waterworks of Port Elizabeth and King William's Town in South Africa, the Government railways in Jamaica, and many other works. He also founded and worked the Titterstone quarries of the Clee Hill basalt.

Mr. Mackay took great interest in all that concerned his native country and was one of the chief promoters of the Crofters Act. He established and endowed the Bogart Educational Association, and was one of the principal supporters of the Clan Mackay Society, as well as Chief of the Gaelic Society of London. At the General Election of 1892, he contested the County of Sutherland in the Unionist interest. He was a magistrate for the City of Hereford, where he resided for some years prior to his death.

Mr. Mackay was elected an Associate of the Institution on the 14th January, 1873.

⁂ The following deaths have also been made known since the 15th July, 1906:—

Members.

Dredge, James, C.M.G.; *died* 15 August, 1906.

Dyke, Thomas, B.A.; *died* 6 August, 1906.

Elwes, Richard Gervase; *died* 14 August, 1906.

Sharp, Frederick; *died* 22 August, 1906.

West, Henry Hartley; *died* 13 August, 1906.

Associate Member.

Downing, Robert; *died* 9 August, 1906.

Associate.

Mathieson, John; *died* 9 August, 1906.

Information as to the career and characteristics of the above is solicited in aid of the preparation of Obituary Notices.—Sec. Inst. C.E., 25 August, 1906.

Sect. III.

ABSTRACTS OF PAPERS IN SCIENTIFIC TRANSACTIONS AND PERIODICALS.

Suggestions for an Engineer's Transit and Level.

(Engineering Record, New York, 17 February, 1906, pp. 178-81.)

This Paper, by the Associate Professor of Topographic and Geodetic Engineering, University of Wisconsin, outlines a number of important improvements in certain instruments which, as regards radical changes, have not made as much progress as they should have done in recent years. As regards the transit, the Author urges, (1) the importance of increasing the size of the vertical arc, in fact, reversing the relative proportions now obtaining between it and the horizontal circle; (2) the provision of more sensitive plate bubbles; (3) a sensitive level for the vertical circle for the control of its arc; and (4) better ground telescope level vials.

For all these well-considered reasons are given. Magnifying power, chromatic and spherical aberration, definition, flatness of field, illumination and centering of lenses are touched on, and the specification used by the University of Wisconsin is given in full.

There is less to be learnt from the Paper as regards levelling instruments, the American engineer being behind his European compeer in this respect, as the Author admits; nevertheless there are points in this part of the article which are well worthy of attention.

C. O. B.

Hell and Sunnan Railway, Norway.

(Teknisk Ugeblad, Christiania, 1905, pp. 461-3.)

This new line, of which a plan and longitudinal section are given, takes a winding course in a north-easterly direction, skirting the eastern slopes of Stjørdal, Aasen, Trondhjem, Borgen, and Stenkjær fjords in succession. Its southern terminus at Hell is 32 kilometres (20 miles) east of Trondhjem (latitude 63° 30′); and its length to the present northern terminus at Sunnan[1] (latitude 64° 10′) is 105 kilometres (65 miles). The total cost—including land and

[1] A few latitudes are inserted, because some of the places here named are not yet marked on ordinary maps.—A. B.

fencing, cuttings, embankments, bridges, and everything else, down to preliminary work and management—has been about 10·5 million kroner (£583,333), or about 100,000 kroner per kilometre (£9,000 per mile). There are fourteen intermediate stations. The ruling gradient is 1 in 53; and owing to the undulating character of the route there are so many banks as to render the line heavy for working, notwithstanding that the highest point is barely 100 metres (328 feet) above sea-level. The sharpest curves are 250 metres (820 feet, say 12 chains) radius. The longest length of straight line is a stretch of 4 kilometres (2½ miles) alongside Borgen fjord. The rails are 25 kilograms per metre (50 lbs. per yard), except for the first 3 kilometres (15 furlongs) out of Hell, for which they are 30 kilograms (60 lbs. per yard). The six-wheel-coupled locomotives with tenders weigh 60 tons, whereof 29 tons are utilized for adhesion; the maximum wheel-load (*sic*) is 9·8 tons.[1]

In addition to its undulating surface, the clayey nature of the ground presents much difficulty in many places. Landslips have occurred of greater or less extent, which have necessitated considerable labour for draining and otherwise securing the works. There are nine bridges, of which one has two spans of 48 metres (157 feet), another six spans of 35 metres (115 feet), and a third five spans of 115 feet. Soft clay in river-beds was the cause of much trouble with the foundations for the piers; extensive borings had to be carried out for determining the sites. In one place new foundations had to be substituted for those first constructed, which had been shifted by the slipping of the bed. For some of the piers concrete piles were moulded; and for others kegs[2] of concrete were sunk. The ironwork for the superstructures was all of inland make, amounting to about 1,550 tons at a cost of about 420,000 kroner (£23,333). There are three tunnels. The longest is 384 metres (1,260 feet). It was driven by hand-boring at a cost of about 188 kroner per metre (£3 4*s.* per foot), including removal of spoil; in two places, together about 39 metres (128 feet), it had to be lined with side walls of ashlar and arch of clinker bricks, costing about 450 kroner per metre (£7 12*s.* per foot). The big cutting at Vottabakken, north of Stenkjær, amounts to about 125,000 cubic metres (163,500 cubic yards), most of which went to form the embankments adjoining. Half was got out by excavating machines, and the rest by hand-labour. Some of it was used for ballasting; one length of about 30 kilometres (19 miles) was ballasted from a gravel pit. Most of the stations are built of stone, with an upper storey of timber.

The southern half of the line, from Hell to Levanger (latitude 63° 48′), was opened for traffic in the autumn of 1902; and the northern

[1] Presumably a misprint for 4·8 tons per wheel, which on the six coupled wheels would give the 29 tons driving adhesion.—A. B.

[2] The original word is *kager*, "cakes," which looks like a misprint for the Norwegian *kagger*, "kegs." If so, the barrels or casks might suffice for sinking the concrete into the soft clay of the river bed, and preventing it from spreading therein while itself setting hard under water.—A. B.

continuation, through to Sunnan terminus, in the autumn of 1905. The Resident Engineer is Mr. Wæhre. The railway now opened forms the commencement of the great trunk line intended to be carried still further north to Røsvik (latitude 65° 45′) or Bodø (67° 23′). From Sunnan to Røsvik the route will be about 572 kilometres (355 miles), and the cost is estimated at about 50 million kroner (say £2¾ million). A. B.

Canadian Railways and Canals.[1]

(Annual Report (Railways) for the year ending 30 June, 1905.)

The Government railways consist of the Intercolonial and Prince Edward Island Railways, the reports of which comprise the following figures:—

	1905		1904	
Mileage	1,698·26[2]		1,573·26	
	$		$	
Earnings	7,000,853	(£1,458,511)	7,575,677	(£1,849,853)
Working expenses	8,879,291	(£1,849,852)	8,879,291	(£2,121,439)
Nett loss	1,878,438	(£391,341)	1,303,614	(£ 271,586)
Number of passenger journeys	3,046,154		2,887,723	
Goods tonnage	2,856,226		2,057,435	
Train mileage	9,873,594		8,545,743	

Detail of Working Expenses per Train-Mile.

	Intercolonial Railway.		Prince Edward Island Railway.	
	1905 Cents.	1904 Cents.	1905 Cents.	1904 Cents.
Locomotive	42·71	40·24	31·91	37·15
Car and wagon	27·96	26·26	16·51	17·49
Maintenance	23·61	22·94	39·66	33·41
Traffic	12·85	13·50	15·11	14·94
General	7·34	7·96	4·72	5·02

	1905		1904	
Capital paid up	$84,193,087	(£17,540,226)	$78,864,052	(£16,430,011)

Rolling stock, Locomotives	358
,, ,, Carriages	424
,, ,, Wagons	10,782

The Windsor Railway, except for maintenance, is worked by the Dominion Atlantic Railway Company. The Government maintains

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[1] The gauge of the Canadian railways is 4 feet 8½ inches.

[2] The increase is due to purchase of the Canadian Eastern Railway from Gibson to Loggieville, 123·67 miles, and the surrender of the Fredericton and St. Mary's bridge and connected property, 1·33 mile.

the line and receives one-third of the gross receipts. The figures are as follows:—

	1905		1904	
Mileage	32		32	
One-third revenue . . .	$50,039	(£10,425)	$53,634	(£11,134)
Maintenance	$26,863	(£5,596)	$24,281	(£5,059)
Net revenue. . .	$23,176	(£4,829)	$29,353	(£6,075)

The following figures refer to those railways directly controlled by the Federal Government, and others towards the construction of which subsidies have been granted or authorized.

Four railways are referred to as being in arrear in supplying information, and of those, the latest returns of two were for 1903 and those of the two others were for 1904, these statements being used.

STEAM RAILWAYS.

	1905		1904	
Mileage	20,601		19,611	
Do. sidings . .	3,632		3,327	
Earnings . . .	$106,467,199	(£22,180,666)	$100,219,436	(£20,879,049)
Working expenses	$79,977,574	(£16,661,993)	$74,563,162	(£15,533,994)
Nett profit .	$26,489,625	(£5,518,673)	$25,656,274	(£5,345,055)
Passenger journeys	25,288,723		23,640,765	
Goods tonnage .	50,893,957		48,079,519	
Train mileage .	65,934,114		61,312,002	
Paid up capital	$1,248,666,414	(£260,138,838)	$1,186,546,918	(£247,197,274)

Rolling stock, Locomotives . . .	2,906
,, ,, Carriages	2,338
,, ,, Wagons	94,161

ELECTRIC RAILWAYS.

	1905		1904	
Mileage	793		767	
Do. sidings . .	35		33	
Earnings	$9,357,125	(£1,949,401)	$8,453,609	(£1,761,168)
Working expenses .	$5,918,194	(£1,232,957)	$5,326,517	(£1,109,691)
Nett profit .	$3,438,931	(£716,444)	$3,127,092	(£651,477)
Passenger journeys .	203,467,317		180,689,998	
Goods tonnage . .	510,350		400,161	
Car mileage . . .	45,959,101		42,066,124	
Paid up capital . .	$61,033,321	(£12,715,275)	not given	

The total subsidies to all railways amounted to the 30th June, 1905, to $32,617,561 (£6,795,325).

Reference is made to the exceptionally severe winter and to the dry summer of 1904 in Nova Scotia and Prince Edward Island, when, under the authority of an Order in Council, 36,609 tons of hay were carried free of charge on the Intercolonial line, and a refund of freight charges to the extent of $125,855 (£26,170) was made. The total cost due to these is estimated at $140,000

(£29,166). The report also mentions the abolition by Act of Parliament (Act No. 3, Ed. VII. Ch. 58, of 1903) of the Railway Committee of the Privy Council, and the establishment in lieu thereof of the Board of Railway Commissioners for Canada, which Act came into force on the 1st July, 1904.

The proposed National Transcontinental Railway from Moncton and Quebec westerly through the northern part of the provinces of Quebec and Ontario, and through the province of Manitoba to the city of Winnipeg, and thence to the Pacific seaboard, is referred to, and the terms by which it is to be constructed by the Government and the Grand Trunk Pacific Railway jointly, and to be worked by the latter. C. O. B.

Electrification of the New York, New Haven and Hartford Railroad.

(Engineering Record, New York, 17 February, 1906, pp. 172-3.)

This application of single-phase working is regarded, to a large extent, as experimental, owing to the difficulty of commutation in powerful single-phase motors, the article stating that there is no way to prejudge the matter theoretically, only experience showing whether single-phase locomotives of large power are a possibility. The fact that Mr. Westinghouse, of Messrs. Westinghouse, Church, Kerr and Co., the engineers, stakes his professional reputation on his ability to turn out a good one, is at present the best non-technical argument for the type.

The line is 21·45 miles long, and connects with the direct-current New York Central tracks 11·13 miles long, running into the Grand Central Station, consequently the equipment of the former line must be capable of running on either system. At the power station will be installed 9,000 HP. of water-tube boilers, and four 3,000-HP. Parsons turbines. Single-phase current at 11,000 volts and 25 cycles will be supplied to the main line, and three-phase current for running air-compressors for the signal system, and other purposes. The overhead wires will be carried by bridges spanning all the tracks every 300 feet, each wire being central over the track it feeds, at a height of 22 feet, which, it is believed, will be ample to prevent corrosion by gases from freight locomotives passing below. Details of the overhead work are given.

Electric locomotives are to be used for the present, reasons for which are explained, though multiple unit trains may be used later. Each locomotive unit weighs about 72 tons, and consists of a steel frame mounted on two four-wheel trucks, each axle being driven by a 250-HP. motor. The ingenious form of suspension of these is described, by which not only the weight of the field, but also the armature, is supported on springs. Each locomotive is expected to handle a 250-ton train.

The power house will cost $1,130,000 (£235,416 16*s.* 8*d.*), the

overhead line construction $574,000 (£118,750), and the thirty-five locomotives $1,050,000 (£218,750). Should the single-phase system be unsuccessful, on trial by the first locomotives made, the power house will still be available for direct current operation, and the expenditure for locomotives, making up nearly half the total, will be saved for the purchase of other equipment. C. O. B.

Single-Phase Electric Railway on Long Island.[1]

(Electrical World, New York, 1905, vol. xlvi. pp. 1029-31.)

The Long Island Railroad Company has recently installed a single-phase electric railway of about 5 miles in length, intended to carry passengers from two stations on their main line (Sea Cliff and Glen Cove) to the neighbouring steamboat piers on Long Island Sound. The main feature of the scheme is the equipment of the cars and arrangements for supplying power to the rolling-stock. From the generating-station current is brought at 11,000 volts to a sub-station beside Glen Cove railway station, and is there transformed down to 2,200 volts for supply to the cars. Each car is equipped with two 50-HP. single-phase compensated-series motors, operated permanently in parallel, and designed for current of twenty-five cycles at an effective pressure of 310 volts. There is also on the car an auto-transformer connected between the trolley and the rails, taking the full-line pressure of 2,200 volts and having six taps on its coil from which effective pressures of 160, 190, 220, 250, 280 and 310 volts respectively between tap and running rail can be obtained. The controller has only five running points, which correspond to pressures intermediate between those just named, that is, to 175, 205, 235, 265 and 295 volts respectively. These are the voltages which may be impressed on the motor circuits in operation, and the variation of speed is obtained by changing the voltage supplied. The voltage on the controller studs, intermediate between those at the transformer-taps, is obtained by the use of preventive resistance and preventive reactance-coils, adopted in order to eliminate sparking at the controller. The connections of the controller-circuits with these coils and with the taps on the auto-transformer are shown by a diagram, and are such that the windings on the transformer are never short-circuited, and the circuit to the motors is never opened during the accelerating period. When the controller is in the position giving the highest voltage, the preventive resistance is cut out, and the loss due to the fall of 30 volts across its terminals is therefore eliminated. The auto-transformer is of 50 kilowatts capacity, but is really divided into two separate transformers, each of 25 kilowatts capacity, one placed at each end of the car so as to give better distribution of the weight. W. C. H.

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[1] See also Minutes of Proceedings Inst. C.E., vol. clxiv, p. 426.

Gas-Power for High-Speed Interurban Railways.[1]

J. R. BIBBINS.

(Engineering Record, New York, 17 February, 1906, p. 174–8.)

The combination of gas-power, on a large scale, with high-voltage single-phase electric traction is a precedent in the history of transport, and its application to the Warren and Jamestown railway is the subject of a highly interesting communication.

The line is 22 miles long, and, generally, fairly level and straight, but one gradient ¾ mile long, of 7 per cent. (1 in 14·28) maximum, with a necessary stop at the foot, imposes a severe and fluctuating duty on the power equipment. There are at present five cars, each 52 feet long, seating fifty-nine persons and weighing gross 35 tons, equipped with four 50-HP. Westinghouse single-phase series motors, connected in multiple with duplicate drum controllers. A multi-voltage transformer receives normal line pressure at 3,300 volts, delivering a maximum of 550 volts to the motors. The daily schedule speed is 19½ miles per hour, with stops averaging every 2½ miles, or 30 miles per hour on through trains. All power for the alternating-current line is transmitted at 22,000 volts to two 300-kilowatt transformer stations, at 1½ mile from the respective termini. Here voltage is reduced to 3,300, and delivered to an overhead trolley (*sic*). Current is collected from this overhead by bow trolleys held in contact position by compressed air. Within city limits 550-volt span wire is employed, paralleling the direct-current trolley, the current being collected by ordinary trolley wheels, which are insulated for 3,300 volts for use on the high-voltage trolley, in case of accident to the bows.

There are two main gas-generating units at present installed, with space for a third of equal size, viz., 260-kilowatt 500-B.HP. nominal. Single-tandem units were employed instead of twin-tandem, in order to give greater flexibility of operation. A full description is given of the various arrangements to meet the incessantly fluctuating load conditions. The gas-engine design is characterized by the strong adherence to approved steam-engine practice. The engine operates on the familiar Beau de Rochas or four-stroke cycle, with practically constant quality of mixture. With the tandem cylinder arrangement, one power impulse is obtained on each forward and backward stroke, the resulting crank effort being equivalent to that of a single-cylinder double-acting steam-engine.

Cylinders are 21 inches in diameter by 30 inches stroke, the unit running at a normal speed of 150 revolutions per minute; the engine being actually rated at 470 HP., with a guaranteed maximum of 520 HP.

Full particulars of the frame and cylinder arrangements, of the crank shaft and bearings, and other details are given by the Author,

[1] See also following Abstract.

as well as illustrations, including velocity and pressure curves. The gas is described as uniform in quality, entirely clean, averaging from 100 to 1,000 B.Th.U. total, and costing 15 cents (7½d.) per 1,000 cubic feet. This placed the cost of power so far beneath the usual figure, that any other source of motive power was out of the question. A long Paper terminates with a Table of work and consumption on the interurban system only for December, 1905, and for the interurban and city systems for the week ending 12 January, 1906, from which the following more important figures are given:—

	Interurban.	Interurban and City.
Number of cars (35-ton)	3	3
Do. city (10-ton)	..	7
Gas per day (average) . cubic feet	54,000	89,285
Hours operated per day	17·5—18	18
Cost of fuel per hour run . . .	$0·464 (23*d.*)	$0·744 (37·2*d.*)
Do. per car-hour	$0·155 (7·75*d.*)	$0·0744 (3·72*d.*)

C. O. B.

Warren and Jamestown (U.S.A.) Railway Power-Station.[1]

(Electrical World, New York, 1906, vol. xlvii. pp. 363–6.)

The Warren and Jamestown (Ohio) railway system consists of a single-phase electric railway 22 miles long, and 9 miles of single track in the City of Warren, which were previously equipped for direct-current traction, and are now to be used both for local service under the old system and for through trains on the new system. The main feature of the power-station is that gas-engines are exclusively used to drive the engines. The new driving-plant consists of two Westinghouse gas-engines of the single-tandem type and rated each at 470 B.HP. The engines work in parallel on the station load, each being directly connected to a 260-kilowatt 25-cycle alternator developing 380 volts. The current is led to oil-immersed transformers, where the pressure is raised to 22,000 volts for distribution to sub-stations, where it is again reduced to 3,300 volts. At this pressure current is supplied both to the trolley-line for the single-phase motors, and to auto-transformers along the city section of the track, which reduce the pressure to 550 volts for use within the limits of the terminal cities. The transformer-stations are operated entirely without attendants, and only require occasional inspection. In the City of Warren, where the two systems—alternating- and continuous-current—are used on the same lines, the two trolley-wires are suspended side by side from the same spans. On the high-pressure part of the system the bow-trolley is used on the cars, but on the low-pressure terminal sections a change is made to the wheel-trolley.

The choice of gas-engines for the central station was largely due

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[1] See also preceding Abstract.

to the abundant supply of cheap fuel-gas in the neighbourhood: and only natural gas is used. The supply is very uniform in quality, very clean, and costs 15 cents (about $7\frac{1}{2}d.$) per 1,000 cubic feet. The engines are started by compressed-air supplied from storage-tanks in the basement. In the engine-cylinders there are valve-chambers at each end for vertical access to the interior. By removing a valve at each end of the cylinder, access is had to the inside through wide openings for cleansing purposes. The Paper contains a full description of the valve mechanisms of the engines and of their operation and control. It also gives the details of the equipment of sub-stations and line equipment.

W. C. H.

Single-Phase Locomotive. A. Solier.

(Éclairage Électrique, Paris, 1906, vol. xlvi. pp. 21-4.)

The Author describes the equipment of one of two electric locomotives which have been under trial recently on the Swedish railways with a view to the eventual electrification of all the lines on the single-phase system. The locomotive, which weighs 36 tons and is designed to draw goods-trains at 28 miles per hour on the level and 15 miles per hour on a gradient of 1 in 10, has three axles, each of which is driven through gearing by a separate compensated series motor of 110 HP. with single-phase current at 320 volts and 25 periods.

The transformer is carried on the locomotive in a compartment behind the driver's cabin. It is oil-cooled and is designed for a normal power of 300 kilovolt-amperes. The primary, which is supplied with current from the line, can take current at different pressures, ranging from 5,000 volts up to 20,000 volts by steps of 2,500 volts. This is arranged by forming the primary of eight coils, each wound for 2,500 volts, which can be grouped in parallel or in series. This arrangement has been adopted to allow of experimental determination of the limit to which in practice the line pressure may be raised with security. In actual trials the pressure supplied from the station may be given the range of values above-mentioned and the necessary grouping of the transformer coils is made on the locomotive before each test. The secondary winding consists of a main coil and several auxiliary coils by means of which the pressure at the motor-terminals may be varied, by steps of 20 volts, from 160 to 320 volts. There are also two sets of terminals corresponding to pressures of 120 and 240 volts, the former for the lighting and heating of the locomotive and for the operation of auxiliary motors at half-speed, and the latter for supply to these motors driven at full speed. Each of these auxiliary motors is a series motor of 7 HP., and one drives the pumps for the vacuum-brakes, while the other drives a suction-fan for air-circulation through the traction-

motors, a pump for circulation of the oil of the transformer bath through cooling tubes, and a pump supplying air, compressed to 6 atmospheres, for the whistle. The current is collected by two aluminium arcs or bows, specially designed to give the same pressure against the wire whatever be the height of the wire above the locomotive.

W. C. H.

Trials of Locomotives in Germany. LEITZMANN.

(Verhandlungen des Vereins zur Beförderung des Gewerbfleisses, Berlin, 11 February, 1906, pp. 61–109.)

The Author describes a series of tests of three types of locomotives at express speeds over a track of 151·3 miles in length (243·5 kilometres) between Hanover and Spandau, requiring over 2 hours for the running. These trials were made, as the former experiments on the Marienfelde-Zossen line were considered unsatisfactory on account of the short length of the line and the inaccurate results which were obtained in point of view of consumption and steam-raising capacity. It was found that even in these new experiments 25 per cent. of the total coal consumed was required for starting purposes.

The three test-engines were:—2–5 coupled compound engine, Egestorf type; 2–5 coupled 4-cylinder compound engine, Grafenstaden type; 2–4 coupled engine with steam superheat, Borsig type.

The tests are of an exhaustive nature, and accurate measurements were taken at intervals of one minute during the twenty trials. These trials extended over fuel-consumption, water-supply, steam-production, use of oil and grease, speed and tractive power, etc. Several of the trials failed owing to engine deficiencies, but six complete trials succeeded and are comparable.

The Author has deduced from these experiments the following formulas:—for the indicated power required (L) at the most favourable velocity in which H is the heating surface and v the velocity.

For compound engines

$$L = v + 3\,H + H\frac{v}{25}.$$

For ordinary coupled engines

$$L = 0{\cdot}5\,v + 2{\cdot}5\,H + H\frac{v}{30},$$

v being expressed in metres per second and H in square metres.

The article is accompanied by numerous plates, tables and graphical diagrams.

F. R. D.

Large Steam and Electric Locomotives. J. E. MUHLFIELD.

(Engineering Record, New York, 24 February, 1906, pp. 227-30.)

This is a comparison relating to services on the Baltimore and Ohio Railroad, but, as was elicited in the discussion following the Paper read at the New York Railroad Club, it is not a perfect one, the electric locomotives being not of the most modern type and working in a service that no steam locomotive could handle.

The Baltimore and Ohio Railroad has now in its service six of the former, including a small shunting locomotive; three, having a total weight each of 196,000 lbs. carried on four pairs of driving-wheels, were introduced ten years ago, and two were put into service in September, 1903. These latter were designed as freight-train helping locomotives, each having a total weight in working order of 320,000 lbs. distributed over sixteen driving-wheels, with two rigid wheel-bases of 14 feet 6¾ inches each, and a total wheel-base of 44 feet 2¾ inches. The driving-wheels are 42 inches in diameter, and each pair is geared to a 200-HP. 625-volt motor, with a ratio of gearing of 81 to 19, providing for a total tractive effort, at full working load on eight motors, of 70,000 lbs., and at starting of 80,000 lbs., assuming 25 per cent. tractive coefficient. The engines can move a train of 3,000 (short) tons with a current consumption of 2,200 amperes.[1] These locomotives handle eastbound through-freight steam locomotives, and trains, at Baltimore, from Camden Yard to Waverley, a distance of about 3½ miles, returning light. This distance includes seven curves—some of 5°—seven tunnels 400 to 7,000 feet in length, and grades ranging from 1 in 100 to 1 in 66. The average time passing through is 23 minutes, with a total moving load of 1,660 (short) tons.

The source of power is fully described, and the total running expenses are given as $34·50 (£7 3*s.* 9*d.*) per 100 miles per locomotive.

Some defects are referred to, and the Author enumerates, from his experience in working such machines, fourteen leading requirements, both mechanical and electrical in character.

The steam locomotive with which comparison is made is on a different service. It is of the Mallet articulated duplex compound type on the Mellin system, designed for maximum adhesive, minimum rail pressure per driving-wheel, and long flexible and short rigid driving-wheel base, for mountain grades and curves. The total weight of 334,500 lbs., in working order, is all carried on the 57-inch diameter drivers. The drawbar pull is about 74,000 lbs. when working compound and 84,000 lbs. when working simple. On the ruling grade of 1 in 100 this engine, assisted by a consolidation locomotive, of which the maximum load behind tender is 1,012 (short) tons, can take up 3,210 (short) tons at 10 miles per hour, working in compound gear. The operating expenses are given as $24·50

[1] These two engines were illustrated in the *Engineering Record* of the 22nd August, 1903.

(£5 2*s.* 1*d.*) per 100 miles run, and after making liberal allowances for items not included, the Author decides that there is a large margin in favour of steam working under fairly similar conditions in this case, and, generally, that for handling heavy tonnage for considerable distances the steam locomotive stands still first, when fuel forms the generating power.

C. O. B.

Electric Locomotives for the St. Clair Tunnel.

(Engineering Record, New York, 20 January, 1906, pp. 71-2.)

The working of steam-hauled trains in the long subaqueous tunnel between Port Huron, U.S., and Sarnia in Canada, being found unsatisfactory, electric traction has been decided on, and an informing article describes the conditions of service and the locomotives, six in number, designed to meet them. The tunnel is single, 6,032 feet long, and the portion under the river has a length of 1,718 feet, with an eastward grade of 1 in 1,000. The 2,399-foot grade on the Michigan side and the 1,915-foot grade on the Ontario side up to the tunnel portals are 1 in 50, these inclines being continued to the yards on the American and Canadian sides, at further distances of 2,515 feet and 3,269 feet respectively. These, with the lengths of the level yards, made up a total length of 19,348 feet to be electrically equipped. The conditions of operation are unusual, the trains coasting for a time and then imposing a rapidly increasing strain on the drawbar. After thorough investigation, it was decided that single-phase equipment, without storage batteries, was the best solution. Some idea of the complication arising may be gathered from the fact that the power-station will be supplying power for traction, lighting, and pumping the tunnel drainage, when, if simultaneous, the load on the generators will range from about 800 kilowatts during the 7 minutes the train is coasting, to about 2,000 kilowatts during about the same time that it is on the up grade. The locomotive is to be coupled to its train of 1,000 tons on the level, at a point 1,200 feet from the summit of the grade, and will accelerate this train up to a speed of 12 miles per hour in 2 minutes, when the summit is reached. It will continue to take current until the train is partially on the grade, when it will be shut off and a speed of not less than 25 miles per hour coasting will be attained. As soon as the train is on the practically level track in the tunnel, current will be taken again, and a speed of 10 miles per hour produced for the ascent.

The single-phase locomotive will weigh about 55 tons and will develop a drawbar pull of 25,000 lbs. on a 1 in 50 grade at 10 miles per hour. It is of the rigid-frame type with driving axle boxes held in the same frame which contains the draught gear. It will be mounted on three pairs of driving-wheels, which will sustain the entire weight, distributed by equalizer bars similar to those used in steam locomotive practice. The principal dimensions will be length over end sills

27 feet 9 inches, rigid wheel-base 12 feet, width over all 9 feet 6 inches, height from top of rail to top of cap 12 feet 6 inches, and diameter of driving-wheels 5 feet 2 inches.

A motor will be geared to each axle giving to each unit an aggregate rated capacity of 750-HP. The motors, which are provided with a novel ventilating apparatus, are of the Westinghouse single-phase, alternating-current, series-wound, compensating type, and will weigh each about 14,500 lbs., the armature weighing 5,600 lbs. They are wound for 240 volts. Full details of these are supplied as well as of their attachment between the locomotive frame and driving-axles.

The pneumatically operated pantagraph trolley to collect current from the overhead line, as well as the latter, are well described.

The power plant will include two 1,250-kilowatt, 3,300-volt, 25-cycle, 1,500-revolutions per minute, rotating-field, Westinghouse steam-turbine units, with the necessary accessories.

C. O. B.

Gasoline-Electric Car.[1]

(Engineering and Mining Journal, New York, 24 February, 1906, p. 373.)

The General Electric Company, in conjunction with the American Locomotive Works at Schenectady, N.Y., has recently completed an experimental gasoline-electric car which possesses many interesting features. This type of car is intended to fill the need in sparsely populated districts for a system better suited to the conditions of the locality than the costly steam railway or the electric railway depending on a feeder system. It has the independence of a central station of the locomotive and the low cost and "cruising" capabilities of the electric car.

The experimental car, which made its first long trial run on the 3rd February, is of the combination type, comprising a passenger compartment to seat nineteen persons, a smoking room with seats for twenty-one, a baggage-room, an engine-room, a toilet and a driver's compartment. It is built on the general lines of the standard passenger railway-carriage, and is handsomely decorated. The total length is 65 feet and the weight is 65 tons. There is a complete controller at each end of the car. Two electric motors, each of 200 HP., geared to the driving wheels, receive the current from a gasoline-driven electric generator, a six-pole unit of 120-kilowatts, wound for 600 volts. This generator is provided with commutating poles, by which, in connection with voltage control for regulating the speed, a flexible operating system is obtained. At starting, the field excitation is weak and a large current at a low voltage is required. The generator, while retaining the characteristics of a shunt-wound machine, is separately excited by a 5½-kilowatt, two-pole, compound-

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[1] See two following Abstracts.

wound exciter, working at 110 volts. This is placed on the top of the generator and is driven by a Morse silent chain. By this method of controlling the speed by varying the field strength of the generator, the speed of the engine remains constant after acceleration. The controller is semi-automatic and can be set for any predetermined maximum acceleration. While not designed for high speed, the average running of the car on its trial trip was about 35 miles an hour, a speed of 40 miles being several times reached. The smooth and rapid acceleration formed a noticeable feature of the running, and there was a complete absence of the vibration which might be expected to accompany the use of a gasoline engine. The latter was built for this car by the Wolseley Tool and Motor-Car Company of Birmingham, England, and is considered the most powerful unit yet constructed for this class of work. The Author gives a brief description of the noteworthy features of this engine.

G. G. A.

Gasoline-Electric Car.[1]

(Electrical World, New York, 1906, vol. xlvii. pp. 323–4.)

A short description is given in this Paper of a self-contained railway-car, which has been devised for use in sparsely-populated districts and has recently made a successful trial run from Schenectady to Saratoga, N.Y. The car is made up of an engine-room, motor-man's compartment, luggage-van, and two passenger-compartments, with seats for fifty-two passengers in all. There is a complete controller equipment at each end of the car, that at the front of the train being in the engine-room. The engine-room contains a Wolseley gasoline-engine, developing 140 B.HP. when running at 450 revolutions per minute. It has six horizontal water-cooled cylinders arranged in three pairs, the two cranks of each pair being at 180°. All valves are mechanically operated. The initial charge to start the engine is provided by shells filled with black powder and fired by a hand-trigger. There are two carburettors, each supplying three cylinders. The engine drives a 120-kilowatt direct-connected six-pole generator, designed for 600 volts and having commutating-poles. The generator is separately excited by a 5·5 kilowatt two-pole compound-wound exciter working at 110 volts and driven by a Morse chain. The car has two motors, each designed to give 200 HP. The car-speed is governed by varying the field-strength of the generator. In the trials the average speed attained was about 35 miles per hour. W. C. H.

[1] See preceding and following Abstracts.

Self-Propelled Car of the Delaware and Hudson Railroad.[1]

(Engineering Record, New York, 10 February, 1906, p. 164.)

A gasoline-electric car of novel construction is described. It consists of a gasoline-driven electric generator, furnishing current to electric motors, geared to the driving-wheels, and controlled similarly to an ordinary electric car. The car, which holds forty passengers, and is 65 feet long over buffers, and weighs loaded 65 tons (58·03 long tons), has passenger, smoking, baggage, toilet, motor-man, and engine compartments, the latter at one end. A complete controlling equipment is placed at each end. The gasoline engine used is considered the most powerful unit yet constructed for this class of work, developing 160 B.HP. at a speed of 450 revolutions per minute. The cylinders are horizontally opposed, six in number, 9 inches in diameter, 10-inch stroke, all valves being mechanically operated, and the cylinders water-cooled. The difficulty in starting internal-combustion engines of such size has been entirely overcome by using shells filled with black powder to provide the initial charge in one cylinder. On starting the engine, the shell is fired by a hand trigger, the whole being similar to the breech mechanism of a gun. Jump-spark and low-tension ignition are both provided, current being furnished to the latter by a small magneto driven from the engine shaft.

The volatilization of the liquid fuel is produced in two carburettors, which form an integral part of the engine. Each of these supplies three cylinders, and is equipped with two float-feed chambers of the usual needle-valve type. Very flexible arrangements are provided to govern the air supply, so that it may be taken from the atmosphere, or from the crank-chambers, or from both, according to the conditions required. The mixture is heated to the required temperature in a small chamber, which itself is warmed by the exhaust. The lubrication is force-feed for main bearings and pistons, and drip-feed for all other working parts. The gasoline is stored in steel tanks beneath the car, and the burnt gases pass through the roof into mufflers, from which they exhaust into the air. The cooling system for the cylinders consists of radiating tubes on the top of the car.

The transmission is electrical, consisting of a generator and two standard General-Electric-69 railway motors. Current is furnished by a 120-kilowatt direct-connected General Electric generator designed for 600 volts, and provided with commutating poles, by which, in connection with the method of voltage control, a very flexible operating system may be obtained. The advantage of this is shown, and it is explained that, owing to the peculiar operating conditions, the generator, while retaining the characteristics of a shunt-wound machine, is separately excited by a 5½-kilowatt, two-pole, compound-wound exciter, working at 110 volts, driven by a Morse silent chain.

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[1] See two preceding Abstracts.

The trial trip, which was entirely satisfactory, took place on the 3rd February, the average running speed being 35 miles per hour, 40 miles per hour being frequently attained. Smooth and rapid acceleration, and complete absence of vibration, were other features observed.

C. O. B.

Electric Lighting of Railway Trains. ORBAN.

(Électricien, Paris, 1906, vol. xxxi. pp. 17-20.)

The Author describes a system of electric lighting of ordinary steam-driven trains, in which the cars carry batteries only, while one generator, to supply current for the whole train, is placed on the locomotive. A small generator-set, consisting of a small steam-engine coupled direct to an enclosed dynamo, is placed on the top of the boiler of the locomotive just behind the dome. Steam is supplied direct from the boiler to the engine, which is of constant couple and variable speed. The dynamo has a constant field (4 poles) and has no rheostats or brake of any kind. From the dynamo the circuit-wires pass in armoured cables over the tender, and then by flexible couplings the current is led to the first car, and similarly from car to car. In the middle of these couplings there is a "security joint," which yields if strain is put on the wires, and is easily opened if cars or locomotive have to be disconnected from the train. The batteries on the car are sufficient to supply light if a train is left in a station for a time without a locomotive.

The system has been installed on two of the Belgian State trains, giving a rapid service between Brussels and Verviers. Each car is supplied with a simple magnetic apparatus, consisting of an electro-magnet, which only operates at the starting or stopping of the dynamo, and so connects the car lighting-circuit to the dynamo or cuts it out. In the locomotive cabin there are two mechanisms which control the whole system—one, the starter, to admit steam to the engine and so start the dynamo, is operated by the driver; the other comes into play automatically when the tension of the dynamo becomes sufficiently high, and closes the main circuit which supplies current to the electro-magnets on the cars. By this simple method of control light may be supplied to the whole train at any desired moment in daytime—as when a long tunnel has to be traversed—and switched off again, by the act of the engine-driver without calling for action on the part of car-attendants. Arrangements are made, however, whereby light may be supplied to any car by a switch on that car in the battery-circuit. This system, it will be seen, dispenses with generators on the cars, and has the further advantage that all the electrical apparatus requiring supervision is assembled in very compact form on the locomotive, and in a convenient position for examination and repair if necessary.

W. C. H

Electric Lighting of Railway Trains. R. DE VALBREUZE.

(Éclairage Électrique, Paris, 1906, vol. xlvi. pp. 208-16.)

Abstracts of earlier Papers on this subject by the same Author have already appeared.[1] The present Paper contains descriptions of five systems, of which four are discussed in detail. The first system treated is Pieper l'Hoest system, in which the current for the lighting of the whole train is derived from a small generating-set placed on the top of the locomotive.[2] In the Böse system each railway-carriage has a generator of constant power, driven by means of a friction device, which slips when the output tends to exceed a predetermined value. The dynamo is connected in parallel with a small battery of accumulators to supply current to the lamp-circuit and to prevent surcharging of the battery; the switch is so designed that dynamo and battery are directly connected when the lamp-circuit is closed, but are connected through a variable resistance when the lamp-circuit is open. The Denham system is somewhat complicated, and is explained by the Author by reference to a diagram showing all the details and connections. In the Siemens-Schuckert system two generators coupled together are mounted on one axle of each car, and the connections are such that, according to the direction of rotation of the axle, one or other of the generators produces the main current, while the other supplies current to an auxiliary field winding of the first dynamo, and weakens the field of the latter as the speed increases so as to maintain a constant pressure at the terminals. If the direction of motion of the train be reversed, the functions of the two dynamos are interchanged. Another feature of the system is that electrolytic valves are placed in the circuit between each dynamo and the accumulators to prevent discharge from the battery through the dynamo. The Leitner-Lucas system is the last discussed by the Author in this Paper. In view of the severe tests which it has successfully undergone on the Great Western Railway in this country it is explained in great detail. The principle of the system rests on the use of a counter-electromotive force, interposed in the field circuit of the dynamo and varying with the speed of the dynamo. This counter-electromotive force, in the present arrangement of the system, is derived from two supplementary brushes on the armature of the dynamo. When the dynamo begins to run, the current in the field-winding supplied from the auxiliary brushes is in the same direction as that of the main field shunt-winding, but as the speed increases and armature-reaction changes the direction of the axis of the flux through the armature, the potential difference between the auxiliary brushes diminishes gradually to zero, then changes sign, and increases negatively proportionally to the speed. The Author gives a diagram of the connections, and explains, by reference to it, the method by

[1] Minutes of Proceedings Inst. C.E. vol. clxi, p. 370, and vol. clxii, p. 432.
[2] See preceding Abstract.

which the battery is automatically thrown into, or cut out of, the dynamo circuit as occasion requires. He also gives a sectional drawing of the dynamo and curves showing the values of the pressure and current given by this machine at different speeds.

W. C. H.

75-HP. 1,700-volt Direct-Current Motor for Railway Work.

RIKLI KEHLSTADT.

(Schweizerische Bauzeitung, Zürich, vol. xlvi. p. 263.)

The Author points out that since the appearance of the alternating form of motor for railway work less interest has been taken in the design of the direct-current motor, but that Messrs. Rieter and Co., of Winterthur, have for several years been improving the latter type of motor.

For about 2 years they have been testing an experimental motor to ascertain the greatest possible output for a metre-gauge line. This was to develop 90 HP. to 100 HP. with 375 volts, and attracted much attention.[1] Since that date they have devoted special attention to the use of high potential direct current. Until recently 500 volts was considered the highest available pressure, and Messrs. Rieter were the first to build motors for 750 volts direct in the year 1901. The motors were built for the Jorat Railway, near Lausanne, and each developed 40 HP. The same firm has now produced a satisfactory direct-current motor for 1,700 volts, and a detailed description of the design is given, and also characteristic curves for pressures of 1,500 and 1,700 volts, which show that an output of 75 B.HP. was obtained at the higher pressure. As such motors might be used two in series, the insulation was tested with alternating current at 7,000 volts. Under prolonged tests the motor proved sparkless until the output reached 90 HP., which is an overload of 33 per cent. The motor will develop 75 HP. for an hour without heating up to more than 70° C. (158° F.), and 16 such motors are now being built by the firm for the new Bellinzona-Mesocco Railway, where the pressure will be 1,500 volts to 1,600 volts direct current. The Author concludes by remarking that, as it has been proved possible to build motors of 75 HP. to even 110 HP. for narrow-gauge lines to work at 1,700 volts, it seems within the range of possibility to build motors for the ordinary gauge lines to work with direct-current pressures of 2,000 volts to 2,500 volts.

E. R. D.

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[1] See *Schweizerische Bauzeitung*, vol. xliii, p. 192.

Design of Permanent Way for High Speeds. MESNAGER.

(Le Génie Civil, Paris, vol. xlviii. pp. 93–1.)

The adoption of greatly-increased speeds in railway travel has raised the question as to whether the permanent way, which has proved itself quite efficient for lower speeds, is of the best possible design for the altered state of circumstances. This question is of the more importance in view of the fact that there is no reason to suppose that the limit of speed has yet been reached. Mr. Cuénot, having lately gone somewhat exhaustively into the matter, points out that the straining of the road and the consequent loosening of the attachments between rail and sleeper must be due to movements either vertical or horizontal caused by the passing of the load. As regards the latter, experiments made on the Paris, Lyons and Mediterranean Railway show that a comparatively short sleeper (2·10 metres) on being subjected to the load, assumes a curve convex upwards from end to end, while with a longer form of sleeper the curve is sinuous, being convex upwards at the centre and the reverse under each of the rails. Owing to the elasticity of the wooden sleeper the stress on the ballast varies very much from point to point, attaining its maximum under the rail itself and practically vanishing at the centre and ends of the sleeper. The result of this is the altering of gauge, the loosening of attachments and the bruising of the sleeper itself caused by the stress, which was intended to be distributed over the whole width of bearing, being concentrated on one side.

A much more rigid form of sleeper is advocated, and one composed of steel channels with compressed-timber seatings for the rails is described. With reference to the vertical movements, experiments appear to show that the displacement at a suspended joint attains as high a figure as 6 millimetres (0·24 inch), while at the neighbouring sleeper the movement does not exceed half this. Thus the vehicle, instead of passing over an even surface, is continually mounting and descending small slopes. At the joints also it is found that the end of one rail is often appreciably above the adjacent end of its neighbour, with the result that the vertical movement is augmented. In order to remedy this defect it is proposed that the joints should be supported and not suspended.

The general conclusions are that greater rigidity is required, and that this can best be attained by using composite sleepers of large effective bearing area and abandoning the suspended joint.

I. C. B.

Track System of the Philadelphia Subway.

(Engineering Record, New York, 3 February, 1906, p. 139.)

The interesting part of this well-illustrated article is that describing the seating of the rails, which is quite novel. The object is to keep everything in a perfectly sanitary condition, so that the entire road-bed can be washed down with the hose and drained through sumps placed at suitable intervals. No ballast is used, and different forms of construction are employed for the express and local tracks, the rails being 90 lbs. to the yard in both cases. In the latter they are mounted on peculiarly formed cast-iron chairs, which, with the rails, are completely embedded in concrete, of which there is 13 inches under the rail-base. The chairs are spaced 5 feet apart, and are provided, at their upper ends, with guide lugs, in which are adjusting screws. The inside ends of these screws bear against holding blocks which grip the foot of the rail, so that the latter is adjustable to exact gauge by manipulating the screws. The rails are laid in a trench left in the concrete; then, after being correctly aligned, concrete is filled in level with the top, on the outside and inside of the rail respectively. Sumps are provided about 50 feet apart.

In the express road, except at cross-overs, the rails are mounted on yellow pine blocks, 6 inches by 24 inches by 10 inches, to which they are attached by clips and screws. The blocks, 2 feet apart centre to centre, are bolted to steel channel longitudinals 15 inches wide and 12 inches deep, braced at intervals, and filled and surrounded by concrete. There are lateral drains 90 feet apart, through notches in the channels, and the sumps are 500 feet apart. The general design of the subway was described and illustrated in the *Engineering Record*, 25th February, 1905.

C. O. B.

Defects in the Pivot of the Voorne Canal Swing-Bridge, Holland. A. F. de Groot.

(De Ingenieur, The Hague, 1905, p. 15.)

The swing-bridge over the Voorne canal is 144 feet 4 inches long, with arms on each side of the pivot of 80 feet and 64 feet 4 inches respectively, and a clear span under the longest arm of 60 feet. The bridge weighs 183 tons, and rests with a bronze cup on a wrought-iron pillar of $14\frac{3}{8}$ inches diameter. It was opened for traffic in the beginning of 1904, but soon gave trouble as regards the swing motion. It was found that this was owing to the deformation of the friction surfaces of the bronze cup and the wrought-iron pivot post. The bronze should have been of 87 parts copper and 13 parts tin, according to specification, but was found to be composed of 88·35 per cent. copper, 11·66 per cent. tin, and 0·605

per cent. zinc and lead. The casting, moreover, contained numerous blow-holes. Experiments with different compositions proved that after all the prescribed proportions were the best, and gave a material capable of sustaining a load of 18 kilograms per square millimetre (11·612 tons per square inch), without undue deformation or erosion through friction. A new cup, cast according to the original specification, was inserted, and no further difficulties occurred even after 5,000 swings. The Paper is illustrated by drawings.

H. S.

Dødvikfos Bridge: Injection of Cement-Mortar into Cavities in Masonry Piers. — WEFRING.

(Teknisk Ugeblad, Christiania, 1906, pp. 41-3 and 98.)

The Drammen and Randsfjord Railway crosses Dødvik torrent by a girder bridge of three spans—120 and 100 and 60 feet—carried on four masonry piers, of which three are exposed to the strong current. The piers are founded upon rock, and were built during the winter of 1864–5. They are constructed of massive gneiss blocks, bonded and partly held together by iron clamps. The interior of the piers was made up with smaller stones; and all cavities were filled with hydraulic mortar, which was worked with warm water, because the temperature must have been as low as 20° to 30° C. below zero = 36° to 54° F. below freezing. The joints were pointed with cement mortar devoid of lime. In the course of years, with varying height and strength of current, the water penetrated inside the piers, and began to form internal cavities. Attempts were made to remedy this by filling with cement mortar all holes that could be got at, and re-pointing the joints; but the result was not satisfactory. After preliminary experiments upon small walls purposely erected on the spot, injection of cement mortar was adopted at No. 2 pier in April 1904, and again at No. 3 in April and May 1905. Having had charge of these operations, the Author gives a detailed description of the latter work, illustrating it with photographs.

A number of holes were made with the object of opening out all the internal cavities, which were then scoured by a jet of water from a force-pump. One hole, made a little below the top of the ice-breaker (*i.e.* "cut-water") at the upper end of the pier, was found to communicate with a joint near the bottom of the lower end; the intervening distance was about 13 metres (43 feet) with a fall of 3 metres (10 feet). In each of three boxes 4 hectolitres (11 bushels) cement and 4 hectolitres sand were mixed dry and with extreme care; water was added at the last, and the whole was worked till it formed 8·1 hectolitres ($22\frac{1}{4}$ bushels) of uniform paste. Each lot when ready was emptied into a tank large enough to hold a charge of $3 \times 8·1 = 24·3$ hectolitres = 2·43 cubic metres (3·18 cubic yards or 535 gallons or 66·8 bushels). Herein it was kept continuously stirred,

until drawn off by the force-pump, and injected through flexible hose into one after another of the holes. Two of the holes took a couple of charges each to fill them. Great care had to be taken to empty the tank, pump, and hose, after each time of using; otherwise the cement mortar would quickly set hard in them. The pump packings wore rapidly, and required renewal. According to the quantity of cement used, the cavities filled must altogether have amounted to as much as 20·6 cubic metres (26·9 cubic yards). The pier has thus been rendered thoroughly water-tight. The entire cost of the work was:—materials 1,046·14 kroner; wages 1,114·50 kroner; total 2,160·64 kroner (say £120). The pump and tank are not included, remaining on hand in good condition. The mere injection of the mortar does not cost much, when all is prepared beforehand. It took only 32 minutes to inject the charge of 2·43 cubic metres. Hence future work can be started immediately before the spring flood, at which season the mortar can be trusted to harden properly.

Besides strengthening old masonry, the Author suggests that the same method might advantageously be employed also on new masonry which is being carried out in winter, when otherwise the whole of the work would have to be housed over and the house kept warm at disproportionately high cost. This suggestion is controverted by Mr. Arthur Kohmann, who considers that new masonry would take a much larger proportion of mortar than was injected into the old pier, and that the cost would thereby be so much increased as to render the work as cheap or cheaper if done with housing and warming in winter. Moreover on the latter plan the necessary supervision can be exercised for ensuring good work. Other advantages are also urged; and disadvantages attending the injection method are indicated.

A. B.

Danville Arch-Bridge of the Cleveland, Cincinnati, Chicago and St. Louis Railway.

(Engineering Record, New York, 3 March, 1906, pp. 238–43.)

A remarkable double-track bridge entirely of concrete, reinforced, above springing, with Johnson corrugated bars, has been recently completed for the above line. There are one central and two side arches, 100 feet, 80 feet, and 80 feet spans, and 40 feet, 30 feet, and 30 feet rise respectively, the intrados at the crown being level. There is a double grillage of reinforcement, 6 inches under surface at crown and 2 feet at springing, both at intrados and extrados, the longitudinal bars being 1 inch and 12 inches apart, and the transverse $\frac{7}{8}$ inch and about 30 to each arch, spaced closer at the crown. The road-bed stands on a 26-arched viaduct, the piers of which are on the main arches, the thickness of the latter being 3 feet 6 inches for the side spans and 4 feet for the centre. This viaduct is strongly rein-

forced, the vertical $\frac{3}{4}$-inch bars, 12-inch centres, of the piers splaying into the body of the arch below. Full particulars of all the steel-work, of which there are 260,000 lbs. in the bridge, are given, but the most interesting part of the article is the detailed account of the methods of erection by which the monolithic character of the structure, with due provision for expansion joints, is secured. There was some delay after the work reached springing level, but from that point to completion took only from the 4th October to the 17th December, 1905, with one 10-hour shift worked.

C. O. B.

New Portland Bridge. H. A. Crafts.

(Engineering Record, New York, 3 March, 1906, p. 252.)

The building of this bridge, which is 36 feet wide with two 7-foot side-walks, in place of an old one 21 feet wide with two side-walks, involved some remarkable operations, as both a heavy traffic over, including some 750 tramcars daily, and navigation under, were specified to be uninterrupted, though as regards the former, a stoppage of 42 days was subsequently allowed by the city for the substitution of the draw span.

The new bridge is a steel structure consisting of a 200-foot skew span at the west end, a 384-foot draw span, and two 269-foot ordinary spans. The piers of the fixed spans are Cushing piers, the cylinders being 14 feet in diameter up to above low-water mark, and 9 feet above. The centre pier, which is in 60 feet of water at its lowest, is composed of a foundation of 256 piles, driven to depths varying from 33 to 48 feet. The cutting off of these piles was among the more difficult problems of the work. The machinery for doing this, actuated by an electric motor and consisting of a 5-foot circular saw, to cut off the piles to a true level, is fully described. On these, a timber grillage, and on it, 3 feet below low water, a steel cylinder 45 feet in diameter and 30 feet high, filled with concrete and carrying the turntable machinery, were erected.

For the substitution of the fixed spans, a six-post traveller was used. To enable the tramcars to pass, it was necessary to raise the tracks over the false work for the fixed spans some 6 feet, and to provide an incline from the old draw span. The cost was $381,573 (£79,494 7*s.* 6*d.*).

C. O. B.

Rapid Plate-Girder Erection with a Derrick Car.

(Engineering Record, New York, 3 March, 1906, p. 281.)

The rapid replacement of the girders and widening of a New York, New Haven and Hartford Railroad viaduct is a record operation described in this issue. The viaduct included fifteen 90-foot deck spans of old lattice-work construction, which were replaced by plate-

girders, two additional lines of plate-girders being also erected parallel to them for the second track. The latter only appears to be referred to in the article. The contract required completion by the 15th March, 1905, and the company's officers became anxious when in the latter part of February nothing had been touched. The work, however, was begun on the 25th February and finished before time on the 10th March.

The contractors had constructed a derrick car of the following abridged description. It was made of steel with four heavy longitudinal girders, and mounted on two of the heaviest standard-car trucks, with capacities of 70,000 lbs. each. The derrick car weighed 70 (short) tons, and was provided with a locomotive gear, enabling it to travel independently at speeds of 4 miles to 5 miles per hour. The sectional boom of 30 (short) tons capacity could be made 25 feet, 50 feet, or 75 feet long as required, and was easily unshipped for transportation. When in service, it was operated from a heavy A frame without a mast. A two-drum, four-spool Rawson and Morrison hoisting-engine and a 50-HP. boiler worked the machine. Auxiliary weighted booms enabled the derrick to contend against the heavy eccentric loading required to lift the 32-ton new girders from their positions on the cars 18 feet beyond the centre of the derrick car. The derrick car being placed on the new track at the end of the bridge, the girders were delivered to it without being unloaded from the cars on which they were brought from the shops, and were, in the interval between trains, brought alongside on the old track still in service. They were unloaded and set on their permanent seats by single operations, being handled by the boom of the car which advanced on each span as soon as erected. The average time for each girder unloaded and put in place was 25 minutes, and the thirty girders, amounting to 1,000 tons, allowing for traffic interruptions, were erected in 7 days, the maximum being six in an 8-hour day. The force employed was fifteen men, and the completed job took 15 days, while twenty-five men for at least 3 months would have been necessary if ordinary methods had been employed.

C. O. B.

New Facts about Eye-bars.

THEODORE COOPER, M. AM. SOC. C.E.

(Proceedings American Society of Civil Engineers, New York, January 1906, pp. 14–31.)

In the execution of the superstructure of the Quebec bridge, which has a channel span of 1,800 feet, the magnitude of the members and the high working stresses have demanded special study of many points, one of the most important being eye-bars.

The eye-bars forming the tension-members are 15 inches in width, from $1\frac{1}{4}$ inch to $2\frac{1}{16}$ inches in thickness, and of lengths varying from 50 feet to 58 feet. The pins, except in a few special cases, are 12

inches in diameter and from 8 feet to 10 feet in length. The maximum joint has 58 bars on one pin. It was decided that the clearance for the purposes of erection should be $\frac{3}{64}$ inch. The experiments made showed that the bars began to elongate permanently at 12,000 lbs. per square inch, and this elongation increased with increased stress, the permanent elongation under the working stress of 21,000 lbs. per square inch being $\frac{1}{32}$ inch to $\frac{1}{16}$ inch for each eye. The upper part of the pin-holes conformed to the shape of the pin and elongated more than the lower half. The diameter of the lower half decreased by the material closing in under the pull. Outside the pin the metal of the head decreased owing to the compression; the transverse dimensions across the eye and neck were reduced by the flow of metal under tension; the distances on the outside of the sides of the heads parallel to the length of the bars elongated very little.

It was concluded that it is a mistake to use soft and ductile eye-bars, but to get the best results either steel of a higher tensile strength must be used or else the eyes should be stretched longitudinally cold before finally boring to exact length. Since the permanent stretch in different bars pulling on the same pin is different, it was concluded that the elastic stretch and therefore the stresses must vary from about 84 to 107 per cent. of the average stress for bars 50 feet long; the variation would of course be greater for shorter bars.

A. W. B.

Storage and Handling of Truss-bars for the Island Span, Blackwell Island Bridge.

(Engineering Record, New York, 17 February, 1906, pp. 195–6.)

The island span alone, weighing 24,000,000 lbs., of this gigantic bridge forms not only a great engineering work, but even the appliances and methods of handling and storage in connection with its erection excel in magnitude all other work of a similar character.

The span consists of two pin-connected trusses 630 feet long and 60 feet apart, with an extreme height of 185 feet at the ends, and 118 feet in the middle, on centres. The trusses will be connected by upper and lower decks. This span will serve as an anchorage span for the cantilever arms of the main channel spans of 984 feet and 1,182 feet. Some illustrations of these were published in the *Engineering Record* of March 4th, 11th, and 18th, 1905.

Operations at the island were commenced in September 1904, in an enclosure 700 × 240 feet for the false work under the span, and for storage, power-house, shops, etc., the storage area being commanded by two 65-ton electric gantry cranes of 85 feet span and 25 feet clearance above ground.

The power-house was equipped with a 480-HP. battery of

boilers, electric generators and motor, pumps, and an air-compressor to deliver at 100 lbs. pressure through mains extending the full length of the span for riveting, chipping, etc. The steel is shipped from the shop at Steelton, Pa., on platform cars loaded to a maximum of 45 tons, and from these, at Jersey City terminus, to the bridge site on car floats. It is here unloaded by a 65-ton stiff-leg steel derrick of special design, illustrated in the issue above mentioned of March 11th, 1905, and deposited on trucks on a pair of standard-gauge tracks, parallel to the bridge axis, which deliver it to the gantry. The derrick has a 76-foot boom and 60-foot mast, and weighs about 45 tons, exclusive of tackle and hoisting engine, the power of which latter is sufficient to raise 78 tons at the rate of 30 feet per minute.

Each of the gantries is worked by one man, and is equipped with six 50-HP. gas-engine motors, operating three independent hoists at speeds of 8 feet, 15 feet, and 55 feet per minute; the trolley speed is 100 feet per minute, and the gantry travels 200 feet per minute. Each gantry weighs 300,000 lbs. and is designed for shop service ultimately.

C. O. B.

Tube Tunnels in Water-Bearing Ground, and the New Metropolitan Lines crossing under the River Seine.

C. Birault.

(Mémoires de la Société des Ingénieurs Civils de France, Bulletin de Juin, 1905 pp. 852–918.)

This Paper is divided into three parts:—(1) general considerations relating to tube tunnels; (2) a short description of some recent works; (3) the works of the new Metropolitan Lines crossing under the River Seine at Paris.

In the first division the Author describes the progress attained in this class of work during recent years and draws conclusions for use in future practice. The Author deduces formulas for the calculation of the thickness of the walls of tunnels of circular section. These formulas are similar to those which he has deduced for pressure conduits of large diameter.[1] The results obtained from these formulas are stated to give unexpected justification to what is done in actual practice for quite different reasons. It is pointed out that even under the most favourable circumstances, when working in very compact strata and when the grouting behind the tunnel lining is very well done, the stresses are far from insignificant in the case of metallic linings. The Author then indicates a method of approximate calculation leading to simple formulas for circular tunnels. The second part of the Paper consists of a monograph of some recent works. In the third part is given a description of the

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[1] Minutes of Proceedings Inst. C.E., vol. clx, pp. 407, 444.

new Metropolitan Lines and of the crossing under the River Seine, which are about to be constructed. The crossing of the River Seine is intended to be effected by two methods of construction, one by twin galleries of circular section, of 5 metres (16·4 feet) internal diameter, and the other by sinking vertically three caissons in the large arm of the river and two caissons in the smaller arm. At the crossing of the Paris-Orléans Railway the freezing method will be employed. The Author concludes by giving particulars of Line No. 8 (Auteuil-Opéra), which crosses the River Seine twice, and of Line No. 5 (Boulevard de Strasbourg-Pont d'Austerlitz), which runs alongside the Canal St. Martin near the slope on the right bank of the river.

The Paper is illustrated by twelve figures in the text and by two plates. J. M. M.

Detroit River Tunnel of the New York Central Lines.

(Engineering Record, New York, 17 February, 1906, pp. 193–4.)

This subaqueous electric-railway tunnel will be remarkable for its special construction, owing to the requirement that the roofs of the tunnels, which are twin circular ones, shall be practically coincident in level with the bed of the river.

Four designs, A, B, C and D, are proposed for the means of construction, but the first three only differ in detail. Design A provides for: (1) dredging out a trench about 30 feet deep below bed, 45 feet wide at bottom and 70 feet at top or at bed of river. (2) Laying a 2-foot layer of stone at the bottom; then (3) 2 feet of concrete on this; (4) lowering saddles on the concrete to receive the forms; (5) building, floating and sinking into place the wood or steel forms; (6) surrounding these forms with concrete up to the level of the bed of the river; and (7) constructing the reinforced-concrete lining or tunnels proper.

The Author describes in detail how these various operations are to be carried out, and the deviations from them to be permitted to the contractor, especially as regards the deposit, through water, of the concrete.

For designs A, B and C, the tubes, 25 feet apart centre to centre, are 18 feet 6 inches in diameter, with 2-foot 6-inch lining, reinforced near inner and outer surfaces with 1-inch rods 14 inches apart, and hoops same size 6 inches apart outer side, and 12 inches inner side, the concrete being 1 : 1½ : 3½.

Design D is the ordinary shield method, necessitating, however, temporary blankets of clay being first deposited over the line of the tunnels. In this case the tunnels will be 30 feet 3 inches apart centre to centre, and the lining is to be 2 feet thick.

Descriptions of the approaches, grades, curves, etc., are also supplied. C. O. B.

Concrete Beams for Floors and Roofs.

(Ingeniøren, Copenhagen, 1905, pp. 249–51.)

A Danish engineer, Capt. Møhl, has devised concrete beams of T section, which are strengthened by having a couple of longitudinal iron rods or tie-bars imbedded, one above the other, in the bottom of the "stem" or vertical web; and a series of them are laid alongside one another in close contact TTTTT, so that their "flats" or horizontal tops form a continuous floor. Detail drawings are given of beams of 20 centimetres (8 inches) depth and breadth. The thickness of concrete encasing the iron rods is nowhere less than 2 centimetres (0·8 inch). The lower rod runs straight throughout the length of the beam; and its ends projecting beyond the concrete are bent at right-angles for cottering into the walls. The upper rod also runs straight until it approaches the ends of the beam, where it turns upwards into the "flat" top; and its hooked extremities are there imbedded in the concrete. The quantity of concrete is only half of that in a solid beam of same depth and of square section. The beams are simply moulded on the spot between mould-boards of suitable form. The edges of the "flats" are slightly bevelled, so that in juxtaposition they leave narrow longitudinal grooves, which are filled either with mortar, or with a wood liner; where boarding is laid down for covering the beams, it is then nailed upon the liners. Three beams of 8 inches depth and breadth and 16½ feet span, laid side by side contiguously, were tested 29 days after moulding, by loading them up to 120 lbs. per square foot: which is three times their working load, and half as much again as the maximum testing load required by German rules. The deflection was 22·2 millimetres (0·874 inch), and faint hair-cracks could be traced in the bottom edge of the "stem" or web; on removal of the load they disappeared, and next day the beams retained only 5 millimetres (0·2 inch) permanent set. In one of them there were the usual couple of round-iron rods of 10 millimetres (0·4 inch) diameter; in another four of 7 millimetres (0·28 inch); and in the third, eight of 5 millimetres (0·2 inch). Around lift-wells, staircase-pits, and chimneys, the concrete beams are either moulded to the special shapes required, or else are replaced by small iron beams. A photograph shows their application as rafters in a roof. For floors the cost is 2·75 to 3 kroner per square alen (say 8¾*d.* to 9½*d.* per square foot). The concrete beams have been used for a mortuary, and for a new building at the small-arms factory of the arsenal.

A. B.

Design of Concrete-Steel Beams and Slabs. EDWARD GODFREY.

(Engineering News, New York, 15 March, 1906, pp. 290-2.)

The writer's object is to develop practical formulas for the calculations of the strength of beams and slabs of concrete and steel of sufficient accuracy and simple form. He takes the beam as exerting its ultimate strength when the steel is stressed to its elastic limit of 40,000 lbs. per square inch, and the concrete compressed to its ultimate strength of 2,000 lbs. per square inch. From this it follows that the area of the steel should be $1\frac{1}{4}$ per cent. of the area of the beam; and, if the centre of the steel rods be taken at one-eighth of the depth from the bottom of the beam, the ultimate moment of resistance equals 354 $B\,D^2$ inch-lbs., when B, the breadth, and D, the depth, are in inches. For square bars, four diameters apart, the diameter of the steel equals one-twentieth the depth of the beam. Taking the effective adhesion of the concrete to the steel that may be relied on as 50 lbs. per square inch of surface of the bar, it results, for a uniform load, that the diameter of the rod should be one two-hundredth of the span; therefore, from what precedes, the depth of the beam would be one-tenth of the span.

Assuming that the concrete is capable of resisting a shearing force of 50 lbs. per square inch, it is found that the same proportion of depth to span is required. Several examples are worked out.

A. W. B.

Zwoyer Briquette Process.

(Engineering and Mining Journal, New York, 2 December, 1905, p. 1922.)

The Zwoyer briquette process has, after 7 years of experimenting, been brought to a practical issue in the works of the Company in Jersey City and Brooklyn, N.Y. This process is adapted to the agglomeration of any fine material, such as coal-dust, iron-ore, flue-dust or concentrate; but its principal use is with the first of these. In its essential features it consists of a set of mineral rolls, 16 inches by 36 inches, to crush the material to the requisite degree of fineness, and a series of heated furnaces, in passing through which it is dried, heated and mixed with the binder. The materials are introduced at one end of the mixer, where they fall on a rapidly-revolving shaft provided with blades having a slight pitch sufficient to transport them and keep them in suspension in a finely-divided state in the upper part of the mixer. While in this condition the binder is atomized by super-heated steam into the other end of the mixer. The atomized binder has the appearance of a black fog passing through the materials in a direction contrary to their general motion. This action is shown in two sectional illustrations. In this way each particle of the dust is evenly coated with an almost

imperceptible layer of binder, sufficient, however, to cement it into a briquette while hot under the press. Repeated tests have shown that about one-half of the quantity of binder is used as compared with the best processes in Europe on the same material, the briquette being equally hard. From the mixer the prepared materials are conveyed to the press, where, under a pressure of 3,000 lbs. to the square inch for anthracite, and 1,600 lbs. for soft coal, it is formed into biscuit-shaped briquettes 1¾ inch square and 1⅛ inch thick. These are hardened in a drying-oven and subsequently cooled. The briquettes will bear shovelling like coal. They burn evenly, are as hard in the fire as when cold, are odourless, and are completely consumed, there being less carbon left in the ash than in that from natural coal; moreover, they leave but little ash.

The advantages claimed for this process are: (1) low cost of production; (2) the automatic action of the whole plant, which requires only three men to a plant of 10 to 12 tons per hour output; (3) good quality of the product; and (4) adaptation of the process and machinery to any kind of material. The cost of power and labour does not exceed 1*s*. a ton, the cost of the binder being about 2*s*. 3*d*. a ton at New York.

G. G. A.

Anthoni-Prache System of Isolating Foundations against Noises and Tremors. P. Prache.

(Mémoires de la Société des Ingénieurs Civils de France, Bulletin de Septembre, 1905 pp. 346–58.)

The Author divides different kinds of vibrations into tremors and noises, which latter Mr. Anthoni has again divided into two classes according to their mode of transmission. The distinction between these two classes of noises cannot be made by ear. The determination of their origin can alone give information of any value. The interposition of obstacles weakens noises transmitted by air, while the isolating foundations suppress the transmission of noises and tremors through the soil. The various so-called isolating materials are discussed, and the Author shows that gutta-percha alone possesses the three necessary qualities—homogeneity, durability and elasticity, the latter being about twenty times greater than in steel.

The relations existing in the qualities of gutta-percha permit of the establishment of a formula for the speed of propagation of vibrations in that material. From the great difference between the velocities of propagation in gutta-percha as compared with other solids, it results that the intensity of refracted rays is practically negligible, and there is an entire reflection of vibrations when the latter tend to pass from gutta-percha into the soil.

A short description of the method of application and some examples of the system in actual work conclude the Paper, which is illustrated by two figures in the text.

J. M. M.

Timbering of Trenches. F. Lang.

(Deutsche Bauzeitung, Berlin, 5 January, 1906, pp. 10–14.)

The Author describes a new form of timbering trenches in quicksand, running soil, or water-logged ground. He has used the same with success in trenches for sewerage and similar works. Instead of driving wooden sheet-piling, flat or corrugated iron plates, the Author used buckle plates. These plates were made 2 feet broad, 6½ feet to 8 feet long, and $\frac{7}{32}$ inch thick, and were bent to a radius of 27 inches. Double-riveted angle-bars, with two vertical stiffeners, form a driving head for the plates, and the vertical edges are bent to form locking guides, which guide the plates whilst driving and form a tight joint. Two holes are provided at the top of the plates so that they can be drawn after use. These plates after being driven are held in position by means of longitudinal and cross timbers. All the irregularities and play between the timbers and the plates caused by uneven driving are taken up by means of wooden wedges. The cost of such trenching compares favourably with wooden sheet-piling, as these plates are more easily driven, and have the great advantage that they can be used repeatedly. The Author has tried this method in various soils, and has used the same plates more than fifty times without serious signs of deterioration. The article is accompanied by seven figures in the text and by a diagram of relative cost.

F. R. D.

350-*Foot Brick Chimney for Acid Chemical Gases.*

Theodore Lindeman.

(Engineering News, New York, 15 February, 1906, pp. 165–6.)

This chimney, recently completed at a chemical works at Newark, New Jersey, is intended to deal with gases at a maximum temperature of 1,500° F., and is consequently lined with acid-proof firebrick for its whole height. The ground at the site of the structure is soft and marshy. Borings showed that rock existed at a depth of 60 feet below ground-level. The foundation contains 324 piles about 2½ feet apart and forming a square of 45 feet side; yellow pine piles were used of 60 feet minimum length, 14 inches diameter at the butt and 6 inches at the point. The pressure on the foundation piles, with a maximum wind-pressure of 50 lbs. per square foot on the chimney—equivalent to 25 lbs. per square foot on the projected cylindrical surface—is 19 tons per pile on the leeward edge. The thickness of the concrete foundation block is 14 feet, and it is octagonal at the top. The outside diameter of the chimney at the bottom is 27 feet 6 inches and it is tapered uniformly to a top width of 9 feet 10 inches; the thickness of the outer shell being 38 inches at the bottom and 7 inches at the top. The lining is 4 inches thick for the

total height and it is divided into sections of 20 feet in height, each section being carried independently on corbels projecting from the main shell; a clear space of 2 inches being left between the lining and outer shell. The chimney is bound round near the top with a wrought-iron band 3½ inches by $\frac{5}{16}$ inch, and the top is protected by a terra-cotta coping.

A. W. B.

Large Steel-Concrete Chimney Stack.[1]

(Engineering and Mining Journal, New York, 17 February, 1906, p. 317.)

The Author describes a concrete chimney, recently built at the Butte (Mont.) Reduction Works, U.S.A., which he states to be the largest and tallest concrete chimney in the world. The inside diameter, both at top and bottom, is 18 feet, and the height, including the base, is 352 feet 6 inches above ground. On a foundation of slag, poured into position in the molten state, 100 feet square at the bottom, 7 feet below ground, and tapering to about 66 feet square at the top, 11 feet above ground, rests a base of Portland cement-concrete 42½ feet square and 8¼ feet high in the centre. The concrete, as well as the slag, has metal of various forms distributed throughout the mass, some of which projects above the base to help to retain the walls of the chimney. The chimney proper was formed in a system of sectional moulds 3½ feet high, which, when coupled together, formed a complete ring. The walls contain vertical and horizontal steel bars, and rings of 1-inch T-iron wired to the bars. These walls are 18 inches thick for the first 21 feet, and in them are the two inlets, one on each side, of 8 by 17 feet. From the top of the 18-inch wall starts the double shell of the chimney. The outer shell is 9 inches and the inner shell 5 inches thick, the two being separated by a 4-inch air-space, which, at the bottom, is in communication with the atmosphere through port-holes. The inner shell stops at a height of 101½ feet, and the outer shell is continued, 7 inches thick, to the top. For the base 1,000 tons of concrete were used, and for the chimney proper 1,475 tons.

G. G. A.

Concrete Chimney of the Butte Reduction Works.[2]

(Engineering Record, New York, 3 February, 1906, p. 124.)

This chimney, which is the highest yet built in reinforced concrete, rests on a base 42 feet 6 inches square of 1 : 3 : 5 concrete, with two layers of twenty bars each crossing at right angles, and two layers of thirteen bars running diagonally, both being 1¼ inch ×

[1] See also following Abstract.
[2] See also preceding Abstract.

$1\frac{1}{4}$ inch × $\frac{3}{16}$ T-bars. For the shaft there is a double shell 101 feet 6 inches high, with an internal diameter of 18 feet, the inner being 5 inches thick and the outer 9 inches, the space between being 4 inches; the upper shell, 231 feet high, is 7 inches thick, with an internal diameter of 10 feet, a reducing length of 6 feet being interposed. The vertical bars are $1\frac{1}{4}$ inch × $1\frac{1}{4}$ inch × $\frac{3}{16}$ inch T's, 20 inches inner shell; the horizontal rings are 1 inch × 1 inch × $\frac{1}{8}$ inch T's, 1 foot 6 inches centre to centre in inner shell, and 3 feet in outer and single shells. The outer shell and single shell above has 460 bars, reducing gradually to twenty at the top. The concrete in shaft is 1 : 4, and all bars are of mild steel and are figured for working stresses of 16,000 lbs. per square inch.

C. O. B.

Alteration of an Irrigation Weir in the Progo River, Java.

Baron H. G. Six van Otterleek.

(De Ingenieur, The Hague, 7 January, 1906, p. 4.)

For the purpose of irrigating a district in the province of Koedoe in Java, a weir was laid in the Progo river, during the year 1899. It was, however, soon found that the river-bed above the weir silted up to such an extent that the intake sluices above the weir became choked by the great quantities of large pebbles and boulders deposited there at every spate, and especially with a falling river level. To concentrate the force of the current at such times, and to lead it past the intake sluices, the top of the weir was raised in a gradient towards the opposite side. The width of the weir between the side walls is 50 metres (162 feet) in the clear. On the sluice side about 43 feet were left at their original level, and from there the coping sloped up 1 in 20, to 5 feet 4 inches above the original top. The result gave every satisfaction, the current continuing to carry the deposit past the sluices and over the weir, even after the heavier material was no longer in motion in the bed above the weir.

The Paper is illustrated by several drawings and photographs.

H. S.

Proposed Svea Canal.

(Teknisk Tidskrift, Stockholm, 1905, General section, pp. 421–5, 440–2, and 473–6.)

A sketch plan accompanying the first of these three articles shows that the route of the proposed Svea Canal is intended to take advantage of the succession of larger and smaller lakes which intervene between the two Swedish seaports, Stockholm on the Baltic Sea and Gotenburg on the Kattegat. The distance as the crow flies is 240 English miles; and though the winding course

of canal and lakes will be considerably longer, it will save roughly half the length of the present sea-voyage round the southernmost extremity of Sweden. Westwards from Stockholm the lakes to be utilized for the canal route are met with in the following ascending order:—Mälar, Hjälmar, Teen, Toften; these and others are all connected more or less circuitously by rivers, through which they discharge eastwards into the Baltic at Stockholm. In another watershed further west will be utilized Lake Skager, overflowing into Lake Väner (Wener); and nature's outlet from the latter is from its south-western extremity into the Kattegat at Gotenburg by the River Gota, in which occurs the famed Trollhätte Waterfall.

The Hjälmar Canal, connecting Lake Hjälmar with the River Arboga, which runs into the western end of Lake Mälar, was reconstructed in 1819–1830 by J. Edström, who also worked out a project for continuing the canal westwards from Lake Hjälmar up into Lake Väner. Such an extension has been talked of for more than 250 years, and is known as the Svea Canal, because it will run through Svea land. Another canal, already constructed through Gota land, and hence called the Gota Canal, connects Lake Väner through Lake Vetter with the Baltic both at Norrköping and at Söderköping; it has nothing to do with the River Gota. Hjälmar Canal is 13·6 kilometres (8·45 miles) long; for 8·5 kilometres (5·28 miles) it is overland, containing eight locks with a total rise or fall of 21·7 metres (71·19 feet); the maximum at one lock is 3·8 metres (12·46 feet). Each lock chamber is 2·07 metres (6·79 feet) deep, 7·12 metres (23·36 feet) wide in the gate openings, and 35·62 metres (116·8 feet) long between the gates.

Edström's projected extension between Lake Hjälmar and Otterbäcken on Lake Väner, forming the Svea Canal, would be 124 kilometres (77·05 miles) long, of which 50·6 kilometres (31·44 miles) would be overland with locks of the same size as on the Hjälmar Canal. Lake Väner is about 22 metres (72 feet) higher above sea-level than Lake Hjälmar. From the latter the route would be upwards along the course of the River Svart and through Lake Teen into Lake Toften. Here the canal was to diverge into a long loop, by further rising to the north-west through Lake Stora Björken into Lake Möckeln, and thence falling again to the south-west along the River Let and through Lake Skager down into Lake Väner. When the work comes to be done, the loop might be saved by carrying the canal straight across from Lake Toften into Lake Skager over the high ridge intervening between the two watersheds; the summit is 139·9 metres (459 feet) above sea-level, and would be the highest point along the entire route. The rise from Lake Toften would be 65·3 metres (214·2 feet), and the fall into Lake Skager 72·1 metres (236·5 feet). If the depth of the existing Trollhätte and Hjälmar Canals and of the projected Svea Canal were everywhere increased to 6 metres (19·7 feet), as recommended by the Royal Roads and Water Department, the strategic advantages of this inland navigation for the largest steamers and cruisers would be so great as alone to outweigh the augmented cost of the engineering works

A lengthy discussion, with which the three articles are mainly occupied, turns upon the conflicting details of the calculations as to the quantities of water required for feeding the extensive chain of locks along the course of the canal. There is also a diversity of opinion as to the sources from which the requisite water could be obtained, without militating against existing water-rights of riparian land-owners, to whom it is argued that all water pumped up for feeding the locks would be returned as fast as used in the working of the locks. The chief difficulty would occur at the summit level. Here it is thought supply enough could be pumped up by electricity generated by some waterfall in the district, even though at a distance from the canal.

A. B.

Canadian Railways and Canals.

(Annual Report (Canals) for year ending 30 June, 1905.)

By Orders in Council April 27 and May 19, 1903, tolls for passage through any of the government canals were abolished. A sufficient staff has been retained, however, to record the traffic and collect other revenue, such as arises from the leasing of lands, water-power, etc.

The total length of canals is not given.

		1905	1904
		$	$
Revenue (of which $76,546 (£15,991) was for rents)		78,009 (£16,252)	Not given.
Expenditure—	$		
Charged to revenue	354,354		
" " staff	483,824		
" " repairs	413,933		
		1,252,111 (£260,856)	1,099,772 (£229,119)
Loss on working		1,174,102 (£244,604)	

An expenditure of $2,071,594 (£431,582) for construction and enlargement has been charged to capital.

The traffic for the open season of 1903 and 1904 has been as under :—

	1904	1903
Tonnage	8,256,236	9,203,817

Most of the canals are closed from December to May.

Since Confederation, 1st July, 1867, to 30th June, 1905, the expenditure on capital account has been $111,637,259 (£23,257,762) of which $22,342,501 (£4,654,688) has been from the consolidated fund.

The total revenue derived from the canals during the same period was $13,405,516 (£2,792,816).

The report contains a great deal of detail as to the character and amount of traffic passing through the different canal systems, and also gives the minimum lock dimension on each, ranging from 118 feet by 23 feet on the Chambly Canal to 900 feet by 60 feet on the Sault Ste. Marie Canal. On the through route between Lake Superior and Montreal the minimum is 270 feet by 44 feet.

C. O. B.

Report of the Board of Consulting Engineers for the Panama Canal.

(Engineering Record, New York, 24 February, 1906, pp. 199, 200, and 211–18.)

In two articles the rival plans of a sea-level canal, recommended by eight members of the Board, and a lock-canal, favoured by a minority of five, are fully described and discussed, with plans and sections of the former.

As to the sea-level canal, the Board decide that it can be made in about 13 years; that it should be 40 feet deep with a width in rock of 200 feet; a minimum bottom width in earth of 150 feet, and a double tidal-lock at Ancon, of which the usable dimensions shall be 1,000 feet by 100 feet, and a dam at Gamboa for the control of the Chagres River.

After describing the route in detail, which shows the total length, including harbour channels, to be 49·35 miles with 19·17 miles of curves, the article deals with the navigation under the speeds allowable by the cross-section, viz., 6 miles per hour for the largest existing vessel, and 8 miles for the average. The transit, depending on the use or otherwise of the tidal-lock at Panama, according to the tides, will be from 5 to 10 hours.

The cost is given in detail, the total being $247,021,200 (£51,462,750); this includes 20 per cent. for contingencies, administration and engineering, but not such items as military and naval defence, sanitation, lighting, dredges, etc. It is assumed that, including these and settlements with the French company, etc., the total will reach $333,000,000 (£69,375,000).

The harbour-works to be undertaken at each end are then described, with tidal effects at each, and the tidal-lock at Panama. The cross-sections of the canal prism and the question of time of completion are discussed. It is explained that the problem of transportation of material, in the heavy cuts at the summit level, is one of the most important, and it is believed that here 20,000,000 cubic yards of excavation, classed as rock, can be removed annually. The Author proceeds to compare the merits of this scheme as compared with that of a lock canal, giving his evident preference for the former.

In a previous article of the same issue a less detailed description of the lock canal, which President Roosevelt favours, is given. In this scheme 23 miles of Lake Gatun, to be formed by the

proposed Gatun dam, are utilized, and there are several locks of a usable capacity of 900 feet by 95 feet, the summit-level being 85 feet above sea-level. The total length will be 49·72 miles, and the cost is estimated by the minority of the Board at $140,000,000 (£29,166,666). The width varies from 200 feet upwards, and for the depth a minimum of 40 feet is proposed. The time to complete is estimated at 10 to 11 years.

The Board consisted of eight American and five foreign engineers, the latter including the Chief Engineer of the Manchester Ship Canal and the Consulting Engineer of the Suez Canal Company. All of the foreign engineers, and three of the American, including the chairman, voted for the sea-level canal.

Strong articles in favour of the latter scheme appear in the *Engineering Record* of the 3rd, 10th, and 17th March.

C. O. B.

Difficult Excavation on the Hennepin Canal.

(Engineering Record, New York, 10 February, 1906, pp. 151-2.)

This is a ship-canal, and the soil being mostly peat containing quicksand and generally full of water and therefore difficult to deal with, an unusual method was adopted by the Government after the abandonment, owing to the difficulties encountered, of the work by contractors. Two lofty movable timber towers, 57 feet high, were mounted, each on wheels and five rails parallel to, and on each side of, the canal, and at 650 feet apart, out to out. Two 2-inch cableways, 18 feet apart, extended from tower to tower transversely across the canal. Each cableway was equipped with a 1½-yard Heyward orange-peel excavating bucket, which, with its carriage, weighs loaded 7½ tons, and was operated entirely from the tower, so as to descend, load itself, ascend, travel to the spoil-bank, which was between the canal and tower, dump its load, and return for another, without requiring any attention other than is given by the operator in the tower.

The plant was designed to have one bucket to work on each side of the canal, both at the same time with a minimum amount of travel, although, as both are handled independently, either one can operate over the entire length of the span and dump on either side.

Each tower, of which full details are given, consists of two braced together, on the same longitudinal axis, one serving as head tower for one cableway and the other as tail tower for the other cableway. The tower platform, 47 feet by 54 feet, is cantilevered beyond the outside track rail, and counterweighted with about 20 tons of rock-ballast, besides 35 tons of the engine and boiler operating the buckets. This provided for the reactions due to the cables and load. Each tower had a special 12-foot 3-inch by 15-inch double cableway engine, with three tandem 51-inch friction drums, and a 125-HP. locomotive fire-box boiler. The total force, exclusive of track-layers,

was one engine-man, one fireman, and one signal-man for each of the two cableways.

The largest output in 1 month for 1905 was in August, amounting to 48,948 cubic yards, costing 10·7 cents (5·35*d.*) per cubic yard. The lowest cost was in August, 1904, when the output was 51,074 cubic yards at 6·8 cents (3·4*d.*) per cubic yard. This included that of the track laying, which, being constant for any section of canal, would have been lower per cubic yard for a larger section than that of the one described, which is 52 feet at bottom with 2 to 1 slopes.

C. O. B.

Proposed New Form of Lift for Canal Boats. CH. DANTIN.

(Le Génie Civil, Paris, vol. xlviii. pp. 256–8.)

One of the most troublesome problems of inland navigation consists in overcoming great differences of level which frequently exist between two adjacent sheets of water. When a good supply of water is available locks have been almost invariably used, but where this is not the case either vertical lifts or inclined planes have been adopted. The former are impracticable where the difference in level exceeds about 25 metres (82 feet), and they further suffer from the defect that extensive foundations below water-level are required. Inclined planes can only be used where the configuration of the country is suitable, and further, they are somewhat slow in action. The system which forms the subject of the article is thought to possess the advantages both of the vertical lift and of the inclined plane; indeed, it may be described as an inclined plane with a helical path. Attached to suitable upright framing is a path similar to the thread in a nut; on this path are arranged a series of rollers, and on these travel the ends of a girder, from which hang the cage and tank carrying the boat. A particular case is discussed where the form of lift would be especially advantageous, viz., between the Danube and the Oder. The difference in height to be overcome would be 35·9 metres (118 feet) and a 600-ton boat would have to be dealt with. The girder resting on the roller-path is designed to be moved by electric power applied through the medium of a fixed rack. As it would be plainly most undesirable for the vessel to rotate with the lifting-girder, a swivel-coupling is provided between the girder and the cage, the tank and boat being guided by independent framing. The bulk of the weight is balanced by counterpoises. It is calculated that the power required would be about 200 HP., the total weight about 4,200 tons, and the cost 2½ million francs (£100,000).

I. C. B.

Electrical Drainage Works in North Brabant.

(Annalen für Gewerbe und Bauwesen, Berlin, 15 December, 1905, pp. 235–6.)

An account is given of the works in connection with the drainage of various marshes in Holland, in the vicinity of Altona, covering an area of upwards of 11,367 acres. The installation has a normal output of 260 kilowatts, furnished by two similar machines, arranged to work in parallel, and comprises a horizontal tandem compound engine of 200 HP. at 200 revolutions per minute, and a three-phase generator of 130-kilowatt capacity at a tension of 3,000 volts. This generator is direct-coupled to the engine, and an auxiliary dynamo of 9 kilowatts at a tension of 110 volts is used for lighting purposes. There are four Lancashire boilers, working at 150 lbs. per inch, to supply the steam, one always being kept in reserve. From the generating station the current is conveyed to twenty-one pumping-stations, draining areas which vary between 25 acres and 4,200 acres in extent, the lifts ranging between 4·26 feet and 6·56 feet. It would have been too costly a matter to apportion the dimensions of each set of pumps to the work actually to be performed, and it was therefore decided to limit the number of sizes to six types of 2, 5, 11, 23, 46 and 100 HP. respectively. Some account is given of the pumps, and the advantage of the use of electricity for working a number of pumping-stations at relatively small distances apart is insisted on. As compared with steam- combined with wind-power, electricity is decidedly cheaper and more economical, owing to the readiness with which the current can be employed at different intervals to drive a number of small pumps, of which the total output is much below the maximum capacity of the generating-station. A sketch map indicates the position of the marshes and the site of the generating-station and of the pumps connected with it. Incidentally, the question of using the tidal rise and fall, amounting to about 10 feet, for driving turbines and generating electricity at a very small cost, is discussed in connection with the drainage of the marshlands.

G. R. R.

Electric Installation on the Ykade, Amsterdam, for the Royal Netherlands Steamship Company. T. R. van Osselen.

(De Ingenieur, The Hague, 1905, pp. 469 and 539.)

Instead of applying hydraulic pressure in the arrangements for handling cargo on the quays and warehouses, electricity is used, which, with coal at 15*s.* 8½*d.* per ton, gives an economy of 14 per cent. The tension of 550 to 600 volts gave no trouble after the repair of some defective insulations. The Paper gives information in great detail about the whole installation, and is illustrated by numerous drawings and photographs.

H. S.

Cienfuegos Screw-Pile Pier.

(Engineering Record, New York, 20 January, 1906, p. 80.)

The construction of this pier for the Cuban Central Railways is remarkable for the unusual application of screw piles to a rock foundation, this arising from the fact of the piles being ordered, and the contract signed, in view of a different site from that finally adopted, the original one being in sand. The bottom was hard coral rock below 20 feet of water, with an average tide range of 16 inches.

The piles were hollow cast-iron cylinders 30 inches in outside diameter, and $1\frac{3}{8}$ inch thick. They were made in standard 12-foot lengths, connected by bolts through inner horizontal flanges. The lowest section, 27 inches long, was special and provided with a bevelled cutting edge, and with two turns of a screw-blade 5 inches thick at the root. They weighed about 420 lbs. per lineal foot.

At first efforts were made to sink them by the combined use of a pile-driver hammer, aided by a powerful water-jet, and by screwing them down; and the first two piles were sunk 8 feet in about a week, and the next two in 4 days. But for the rest of the work, it became evident that better means were essential.

By the use of a water-jet at about 150 lbs. pressure delivered through a heavy chisel-pointed nozzle, five 3-inch holes for each pile were sunk in the hard rock to a total depth of about 16 feet in half a day, one at centre and at four equidistant points of a 3-foot circle. Through the jet-pipe dynamite cartridges were lowered to the bottom of the holes, the pile was removed, and the wire caught at the bottom and connected to a battery. When these cartridges were fired, the rock was sufficiently broken to enable the pile to be screwed down.

Between 50 and 100 cartridges were used for each pile, and 14 feet was the average depth sunk. A force of twenty men averaged two piles a day under favourable conditions.

Full particulars of the complete pier, including a flooring of reinforced concrete, are given.

C. O. B.

Steam-Pipes in Shafts. R. D. O. Johnson.

(Engineering and Mining Journal, New York, 27 January, 1906, p. 173.)

A steam-pipe carried down a shaft is one of the mining engineering problems commonly regarded as only partially solved. The difficulty arises mainly from expansion. Various kinds of expansion joints are used, depending on the flexibility of coiled or looped sections, or the movement permitted by "fitted" sections, the ordinary gland or stuffing-box. The coiled or looped sections are the most satisfactory in action when they include a sufficient length of pipe; but they take up too much room in a shaft, and they are moreover expensive. The least satisfactory is the gland and stuffing-

box. The Author designed a method for the shafts of the St. Louis Smelting and Refining Company, Missouri, which he states entirely overcame the difficulties. The essential features of this scheme are to throw all the expansion downward and to one point, the bottom; to keep the pipes at all times under a moderate tension; and to relieve the pipes when hot of all stress other than that due to the steam-pressure. To fulfil these conditions the pipes were made up in 60-foot sections, and each section was balanced by a counter-weight. The sections were made up at surface, the flanges being turned, faced and scraped, so that when bolted together they were steam-tight without using gasket of any kind. The pipes were covered with a magnesia non-conductor over which was slipped a tight casing of galvanized, spirally-riveted pipe. At the flanges the casing was crimped over the covering, and all spaces in the crimping were filled with plaster of Paris and tarred to prevent water getting into the covering.

A plumb-line is dropped down the shaft; the timbers intended to support the pipe are marked, and the counter-balancing apparatus placed in position. This apparatus is simply a weighted lever when the total expansion does not exceed 5 inches; for greater expansions a pulley and weight are used. The bottom section is lowered into place, and temporarily supported till the next section is placed, when the lowest counter-balance is attached. The succeeding sections are then lowered and the counter-balances attached till the top is reached. The top section is then firmly clamped to the timbers of the pit-head frame. The counter-balances are placed at every 100 feet. The sections of pipe are weighed before being let down the shaft, and the counter-weights are calculated to balance all but 100 lbs. of the weight of the 100 feet of pipe below. This unbalanced weight keeps the pipe in slight tension at all times. In very deep shafts it would be advisable to use counter-balances at every 50 feet for the first 200 or 300 feet, to prevent too great tension on the upper sections.

G. G. A.

Specific Heat of Superheated Steam. L. Holborn and F. Henning.

(Annalen der Physik, Leipzig, 1905, 4th series, vol. xviii. p. 739.)

The Authors used a calorimeter in which paraffin, at temperatures ranging from 110° to 120° C., was used instead of water. As the superheated steam, which was at atmospheric pressure, could not condense, these experiments are free from the possible error which might be introduced when dealing, as Regnault did, with the latent heat of condensation. Nevertheless, the Authors do not seem to have felt absolute confidence in their results, and having previously determined the specific heat of air, they carried out simultaneous experiments on steam and on air, and with the help of the ratio so

found they estimated the specific heat of steam from their previous experiments on air. The results are as follows:—

Numbers of Experiments. A. S.	Approximate Ranges of Temperatures.	Specific Heats					
		Found for Air.	Found for Steam.			Ratio. Steam / Air	Estimated for Steam.
			Minimum.	Maximum.	Mean.		
17, 17	110—270° C.	0·2315	0·4430	0·4525	0·4492	1·940	0·4623
19, 8	110—440° C.	0·2350	0·4555	0·4635	0·4601	1·958	0·4696
7, 8	110—620° C.	0·2442	0·4683	0·4798	0·4751	1·946	0·4773
5, 8	110—820° C.	0·2492	0·4930	0·5018	0·4978	1·998	0·4859

The Authors compare their estimated results with those obtained by Langen, who experimented with explosive mixtures and high temperatures. His results are represented by the formula

$$C = 0{\cdot}44\ (1 + 0{\cdot}00027/\theta)$$

whereas the Author's formula is

$$C = 0{\cdot}4460\ (1 + 0{\cdot}000{,}096\ \theta).$$

Here C is the mean specific heat between the temperatures 0 and θ. As will be seen, their estimated high temperature values are lower than Langen's. The discrepancy would have been considerably reduced if they had adopted the actual results of their own experiments in column 6.

C. E. S.

Steam-Consumption of Reciprocating Engines.

T. Stevens and H. M. Hobart.

(Electrical World, New York, 1906, vol. xlvii. pp. 369–71.)

The Authors have obtained from different representative British firms of builders of reciprocating engines, and from the published results of engine tests, a large number of data relating to steam-consumption. By analysing these and by plotting curves they have endeavoured to find representative curves for engines of the type considered. The standard basis of reference adopted is an absolute steam-pressure of 185 lbs. per square inch with 50° C. (90° F.) of superheat and a vacuum of 86·6 per cent. (26 inches). Representative curves for engines working under these conditions are given for full load, half load, and quarter load, and then by analysing further data, other curves are obtained from which an investigation of the effect on steam-consumption of admission pressure, superheat, and vacuum may be made. From the mean curve representing standard conditions there can thus be obtained

other curves showing the steam-consumption of piston-engines designed for other conditions. The Authors give a series of such curves showing steam-consumption under full load.

W. C. H.

Relative Economy of Steam-Turbines and Reciprocating Engines.

T. Stevens and H. M. Hobart.

(Electrical World, New York, 1906, vol. xlvii. pp. 410-12.)

In this Paper the Authors, making use of published data on the performance of the steam-turbine, prepare representative curves of steam-consumption for this type of engine similar to those they have already given for reciprocating engines.[1] They next proceed to compare the two sets of curves in order to arrive at the relative economy of the two types of engine under different conditions of admission pressure, superheat, and vacuum. In this comparison the Parsons type of turbine is alone considered, as the data and test results on this type are more exhaustive than on any other. The curves of full-load steam-consumption for the two types of engine show that with high vacuum and low admission pressure the turbine has an advantage over the reciprocating engine. The use of superheat has a much greater influence on steam-consumption in the case of reciprocating engines than in that of turbines; but, on the other hand, high vacuum has a more beneficial effect on the economy of the turbine than on that of the piston engine. The turbine, again, is but slightly dependent on admission pressure. The question of oil economy is also referred to. Where coal can be obtained at 12*s*. per ton, the cost of oil will generally amount to about 8 per cent. of the cost of coal in the case of good modern reciprocating engines. In the case of turbines the outlay on oil is said to be reduced to an exceedingly low figure—from 0·5 per cent. to 2 per cent. Thus if the figure be taken at 3 per cent. for a district where coal costs 12*s*. per ton, there remains an advantage for the steam-turbine of 5 per cent. on the combined outlay for coal and oil.

W. C. H.

Gas-Turbine. A. Berthier.

(Le Génie Civil, Paris, vol. xlviii. pp. 72-4.)

The development of the internal-combustion engine and of the steam-turbine has been very rapid during the last few years, and it would at first seem that the great advantages which characterize both might be readily combined in a gas-turbine. Against this

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[1] See preceding abstract.

there are three principal difficulties: (1) securing adequate compression; (2) overcoming the deleterious effects of the excessively hot gases; and (3) the great loss by radiation. The earlier turbines of the type in question might more properly be termed hot-air machines, as their action depended on cold air being compressed in a separate chamber where it was expanded by being heated from the outside and was subsequently made to impinge on the blades of the turbine. More recently the fuel, in the form of either gas or the vapour of one of the liquid hydrocarbons, has been introduced into the chamber with the compressed air and the mixture kept burning continuously, thus expanding itself as does the charge in an ordinary gas-engine cylinder. The heat thus produced is somewhat excessive, attaining a temperature of 1,800° C. In turbines recently constructed the combustion-chamber is cooled by a limited quantity of water, which, by the heat, is converted into steam and then made to impinge on the blades of the turbine, either by means of a separate jet or by being allowed to mix with the burnt gases issuing from the combustion-chamber. Thus the gases after combustion are kept cool, and the energy which would otherwise be lost is returned to the machine by the use of the steam-jet. The solution of the problem of the gas-turbine is by no means complete, and the value of the above-named systems can be determined by experience alone.

Stress is laid on the fact that the efficient working of a turbine depends on the velocity of the fluid which acts on the blades. The case of a gas-turbine differs from one driven by either steam or water, as in the latter case the velocity is due solely to the difference in pressure between the water as it enters and as it leaves the guide-blades or jet, while in the case of gas or steam a further factor has to be considered, viz., expansion, and in the case of steam change of state plays a not unimportant part. The effective velocity is thus due to either mechanical or thermic causes, and these may act either in conjunction or in opposition. Although the behaviour of steam or gases when acting on a piston has been exhaustively studied, little is known of their action when expanding more or less freely, and it is suggested that the subject is one worthy of the attention of the physicist.

I. C. B.

New Coal-Tips at Garston Docks.

(The Engineer, London, 16 March, 1906, p. 279.)

Two powerful hydraulic coal-tips, each weighing about 200 tons, have been recently erected at the docks at Garston, near Liverpool. The tips are made of steel throughout, and are provided with eight flanged tread wheels to run on rails about 26-feet 4-inch centres, the extreme wheel-base being 39 feet, and the height 70 feet. They are designed for lifting or lowering a coal-wagon of gross weight up to 30 tons, through a height of 30 feet. The

wagons are taken on and off at a level of 25 feet above the quay level, and can be tipped and discharged at any level between 5 feet and 35 feet above the quay level. The lifting-platform is 26 feet long and 14 feet wide, and is adapted for any size of wagon up to 24 feet 6 inches long over the buffers. It is raised by four direct-acting lifting-rams, two, 9⅛ inches and 6⅛ inches in diameter respectively, being placed on either side of the structure; they are attached at their upper ends to a crosshead from which the platform is suspended by two steel lifting-rods. Their valve gear is so arranged that the smaller or the larger cylinders can be connected to the pressure or the exhaust to enable the tip to lift or lower a loaded or empty wagon with the greatest possible economy of water.

The cradle on the platform is tipped by a hydraulic jigger carried on one side of the structure, having two steel wire ropes, each of which is anchored at one end to the cradle and the other end to the platform, so that they travel with the platform when lifting or lowering, the cradle remaining horizontal until the jigger is put into operation, when the ropes are taken up as required to tip the cradle the necessary amount. The tips are provided with a hydraulic engine for traversing either way along the rails at a speed of 50 feet per minute; this engine also operates the winch for inclining the shoots. A. W. B.

Hydraulic Coal-Tips and Electric Cranes at Doklaan Station, Amsterdam. G. H. Brandt.

(De Ingenieur, The Hague, 1905, p. 420.)

The new loading and discharging installations at Doklaan Station include two hydraulic coal-tips and eight electric cranes, as also three steam and twelve hand-cranes. The ground level being only about 4 feet above the water in the canals and harbours, the coal-wagons have to be lifted about 6 feet before the contents can be tipped on the barges alongside. This is done by hydraulic pressure furnished by a horizontal engine of 75 I.HP.

The current for the electric machinery is supplied by the town council. The Paper gives a very complete and minute description of details and is illustrated by a great number of drawings, diagrams and photographs. H. S.

Coal-Crusher. C. Eitle.

(Journal für Gasbeleuchtung, Munich, 10 February, 1906, p. 129.)

The jaws of the crusher are set inclined to each other at the lowest point; a conical space thus remains between, which serves for the reception of the material to be broken. The machine is so

geared that these converging jaws are made to rise and fall in their respective planes. The distance between the jaws is augmented by the upward movement and the coal slips down on the teeth; by the downward movement the distance is reduced and the coal is broken. The teeth are of prismatic form and run in a vertical direction, except at the lower part of one jaw, where they are set slightly askant, otherwise they are exactly opposite one another. By this arrangement the coal is split in one direction in the upper part of the machine and in another direction in the lower part. The Author claims to be able to reduce coal to pieces of approximately equal size.

In order to reduce friction the supports of the jaws are placed on rollers which move on steel plates fixed to the frame of the machine. Those parts subjected to wear and tear are easily renewable.

The crusher has proved itself efficient with the hardest of coals, viz., Scotch cannel and Australian boghead. For breaking 30 tons of English anthracite per hour 6 HP. is required. The machine is made in various sizes to deal with 5 to 50 tons of coal per hour.

E. V. E.

Turbine Pumps for Mines.

(Engineering and Mining Journal, New York, 14 October, 1905, p. 687.)

Several turbine pumps have been successfully working during the last 2 years in the anthracite coal-field of Pennsylvania; and now a number of new installations under all conditions, paralleling in some instances plant of other types, are being made. H. R. Worthington in America and Jaeger in Germany have in the last 3 years greatly improved the turbine pump, which has now reached a high degree of efficiency. Originally water was delivered into a tank 100 feet above the turbine; then another lifted this water 100 feet higher to a third pump, and so on. The next step in advance was to connect the water-column of one pump with the suction of the pump above. Later this form of connection was done away with. As now built the impellers are on the same shaft, and the discharge of one impeller is connected with the suction of the next, all in one casing and on the same bed-plate, forming practically one machine and requiring only one motor. These pumps are built to work at any angle, so that in unwatering mines they may be applied to shafts, slopes and floating stages. A practical advantage possessed by the turbine is that it will pass without trouble any foreign material which can enter the water passage through the impeller. Also the velocity of the water in the suction and the discharge is constant, and consequently no shock is felt in the pipe.

One of the highest turbine lifts in successful operation in America is at Butte, Mont. The plant there consists of two units, each capable of delivering a maximum of 1,000 gallons a minute against a head of 1,350 feet, the lift being accomplished in twelve stages. Among

the turbine pumps in operation in the anthracite mines is one at a colliery of the Delaware and Hudson Co., which has been working 2 years in acid mine water. A special composition was used for this, consisting of seven parts copper and one of tin. After 2 years' working the tool marks are still visible in the metal. At the Avondale mine another turbine pump is being installed capable of raising 2,800 gallons a minute against a head of 100 feet. This pump has a vertical shaft and is capable of running submerged. The motor is placed 90 feet above the pump. Among the Hampton plant is a turbine pump designed to lift 5,000 gallons a minute against a head of 500 feet, and driven by an 800-HP. motor; and another of 1,000 gallons capacity against a 600 feet head. A good illustration of what can be done by turbine pumps is afforded by the case of a mine which is about to be unwatered. The workings are said to contain 2,000,000,000 gallons of water. There are two shafts and two slopes on the property. The former are 800 feet deep, and the latter 400 feet on a steep pitch. It is calculated that to do this work, eight 12-inch turbines of the four-stage type, raising each 4,000 gallons a minute, will be required. Down to the 400-foot level two pumps will be placed on each slope and two in each shaft. It is estimated that with this plant the water can be removed in 85 days.

G. G. A.

Pit-Head Frames. R. D. O. Johnson.

(Engineering and Mining Journal, New York, 24 February, 1906, p. 364.)

Pit-head or gallows frames for shafts are usually constructed with inclined cross-braced stiffening legs to give stability to the structure by resisting the overturning moment due to the pull of the rope while hoisting. In calculating the weight of a well-designed 75-foot frame for a two-compartment shaft, 60 per cent. of the total weight was represented in these legs. Another disadvantage attending their use, sometimes serious where crushing, screening or conveying machinery has to be installed, is the space they occupy around the pit-mouth. Seeing that it is more economical of weight to use tension members in place of struts, the Author designed a pit-head frame retaining only the vertical portions of the usual design, and substituting wire-rope guys to resist the overturning moments. This frame, the total cost of which was only about one-half the tender for a corresponding design with the usual inclined legs, has been in constant use for the last 6 years, and has proved satisfactory. Moreover, the space between the shaft and the engine-house is left clear for such apparatus as may be required. A sketch is given of this guyed design, in which four $\frac{3}{4}$-inch guys are shown anchored to "dead men." After they have been drawn equally taut, $\frac{3}{8}$-inch bridle ropes, one from each of the guys to the vertical frame, are fixed and tightened. The use of these bridle ropes is to check the tendency of the guys to slap in consequence of the irregular stresses

to which they are subjected in hoisting. This frame is steady at all times, and has shown no movement under any test hitherto applied. The resistance to overturning moments is divided between the guys, the capacity of the vertical members to resist eccentricity of loading, and the uplift on the foundations. The guys should be depended on solely for resistance to overturning; and the foundation should be no more than is necessary to support the vertical members, the load they may have to carry and the pull of the guys. In accordance with these conditions, and with a view to a further saving of material, the Author shows an improved design of the vertical portion of the frame.

G. G. A.

Core-Drilling without Diamonds. LUCIUS I. WIGHTMAN.

(Engineering and Mining Journal, New York, 4 November, 1905, p. 830.)

The now well-known system of core-drilling without diamonds consists, in its original form, essentially of a steel cylindrical shell having a series of chisel-edges ground or forged at one end, the other end being threaded for attaching to the core-barrel. The cutter and barrel are rotated by the drill-rods. These rods are hollow, and through them a stream of water is constantly forced downwards to the bits. This stream carries the cuttings up at a high velocity to the top of what is called the "calyx," where the opening is increased. The velocity being thereby reduced, the cuttings settle into the space between the calyx and the rods, where they accumulate in a second but reversed record. This form of cutter will rapidly and cheaply cut the majority of rocks; but there are some too hard to be cut by it at an economical rate. When such rocks are met with, a newer form, known as the "chilled-shot method," is used. Chilled shot, which is simply molten iron or steel atomized and suddenly cooled, will scratch glass. The shot-bit is of a special metal, and it simply rotates in the bore-hole. The chilled shot is between the bit and the rock; and under the rotation and pressure of the drill-rods the rock surface is ground or crushed away. There is no rock hard enough to resist this action; and all can be bored by this process at a payable rate. The shot is cheap. Its rate of consumption varies from 10 lbs. to 50 lbs. per 100 feet bored.

Another improvement recently added to these prospecting-drills is a double core-barrel specially designed for securing cores from soft coal. The prospecting of soft-coal fields by core-drilling has always been a matter of difficulty. The jar and vibration of the ordinary core-drills shatter the core, and the constant grinding of the tube on the core reduces the coal to powder, thereby destroying any reliable record. With this new device 95 per cent. to 100 per cent. of perfect cores have been brought up. The Author shows the con-

struction of the new double core-barrel in a sectional illustration, and describes the working of the several parts. The device consists essentially of an outer rotating barrel carrying the shot-bit and an inner stationary barrel acting as a protecting sleeve round the core, which is simply forced upward and not turned round, and thereby ground into fragments. A row of steel balls near the lower end of the inner core-barrel reduces friction and keeps the barrels concentric, and another ball-bearing near the top of the drill serves to steady it, and so prevent the core from being broken by the wabbling of the drill. The value of this new system of core-drilling in soft materials has been demonstrated in practice. The largest hole provided for in the standard drill of the Davis system is $10\frac{1}{4}$ inches diameter, yielding a 9-inch core; the smallest is $2\frac{5}{8}$ inches, yielding a $1\frac{5}{8}$-inch core. A depth of 3,000 feet can be bored without special equipments.

G. G. A.

Shaft-Sinking in Wet Ground by Cementing. A. Habets.

(Revue Universelle des Mines, Liége, December, 1905, p. 237.)

At Béthune, in the Pas de Calais coal-field, a method of sinking by hand through heavily-watered strata, suggested by Mr. H. Portier, has been successfully applied in a new shaft, No. 11 of the Béthune Colliery Company, where the coal measures are covered by about 300 feet of cretaceous strata, including a considerable thickness of chalk with flints, which is full of open water-bearing fissures. The essential principle of the method is the injection of liquid cement into the rock by means of bore-holes within the ground to be excavated, which, when the cement has had time to set, is sunk through by hand until an impermeable stratum suitable for the seat of the tubbing is reached. The work was begun by sinking and heavily walling an advance pit, 5·2 metres in diameter, to a depth of 4·6 metres. From the bottom of this four bore-holes, spaced equally apart on opposite diameters of the pit and about $4\frac{1}{2}$ inches from the side, were put down to the gault (Dieves), a depth of 95 metres. Two of these were bored by Mr. De Hulster by the method of alternate boring and sludging, and the other two by Mr. Raky, with continuous flushing of the detritus. The borings, which are only lined near the surface, are closed at the tops with covers having a packed joint for the passage of the central injection tube. In the De Hulster method the cement injection, like the boring, is intermittent, and is repeated with every fresh depth of about 8 metres bored, the excess of cement returning to the mixer by a special pipe. In the Raky system the liquid cement replaces the flushing water, and is injected by the hollow boring-rod, the air escaping with the sludge at the surface. A small Burton pump, capable of giving a pressure of about 60 lbs. to the square inch, was used for the injection. Until the water-level was reached at about

45 metres the cement used was of its natural grey tint, but below that, in order to trace the distribution from the different bore-holes, it was artificially coloured, red being used in the De Hulster and black in the Raky borings. The boring (95 metres) was done between the 15th May and the 22nd July, 1904; the cement injected amounted to 1,776 sacks, or about 87 tons. On the 17th October the sinking was commenced in the ordinary way, and continued until the 1st December, when the seat of the tubbing was reached in the clay below the chalk. Cement was first noticed at 33½ metres in small cracks in the fissured chalk, the filling being still soft three months after the injection, and similar conditions prevailed more or less down to about 60 metres, where the fissures were found to be well filled, and the cement hardened. At greater depths, however, the filling diminished, and the fissures between 70 and 90 metres gave out considerable quantities of water, from 20,000 to 30,000 gallons in 24 hours. The cement seems to have been most efficacious when applied intermittently, as in the other system; its binding power was diminished by the mixture of mud from the boring, with the water. A similar effect seems to have been produced by the clay in the marly beds at the bottom of the chalk. The pit is lined with cast-iron tubbing between the levels of 95 and 33 metres, and from the latter point to the surface with brickwork.

The sinking through the coal measures was done by the Sullivan diamond drill and by blasting with dynamite, from twenty-eight to thirty-two holes being bored at a time, and fired in four volleys, the sinking and walling being carried on alternately. A double cylinder, 24-inch by 6½-foot stroke engine, with safety catches, was provided for hoisting the broken rock, and a special capstan for the walling platform, which was made in two sections, to suit the cage compartments, with a further division to allow the parts to pass through the holes for the sinking tubs at the surface, which were made as small as possible for reasons connected with the ventilation.

Between the 17th October, 1904, and the 31st July, 1905, a period of 8 months, the pit was sunk to a depth of 400 metres. In the first period the daily rate of sinking in the soft ground was 2·62 metres, and that of building in the tubbing 2·35 metres. For the whole depth of 400 metres the averages, including the tubbed portion, were:—

Sinking	2·16 metres per day.
Lining	4·87 „ „ „
Sinking and lining	1·42 „ „ „

a speed which has probably only been obtained previously when the two operations have been carried on simultaneously.

H. B.

Splitting Rock in a Quarry by Air-Pressure.

(Engineering News, New York, 1 March, 1906, p. 248.)

The granite of the North Carolina Granite Company of Mount Airy covers a gently sloping hillside, and consists of a homogeneous mass of moderately hard granite which shows no ledges or bed planes, but splits readily in a straight line in almost any direction. This property is taken advantage of to create artificial ledges by separating the rock along approximately horizontal planes. This mass is then broken up into suitable sizes.

In commencing operations a drill-hole about 3 inches in diameter is sunk to a depth of 6 feet or 8 feet, according to the thickness of stone required. The bottom of the hole is chambered out by exploding half a stick of dynamite there, after which a charge of about a handful of black powder is exploded in the pocket so formed, starting a horizontal crack. Successive charges of increasing quantity are exploded until the cleavage has extended to a radius of 75 feet to 100 feet in all directions. A pipe fitted with a valve is next cemented into the drill hole, and, after setting has taken place, is connected by a pipe to an air-compressor. The air is gradually admitted at a pressure of 70 lbs. to 80 lbs. per square inch until the crack extends to the face of the slope, only about half an hour being required to separate several acres of granite in this manner.

A. W. B.

Meurthe and Moselle Coal-Basin. L. Bailly.

(La Nature, Paris, 23 December, 1905, pp. 50–1.)

In order to explore the further extension of the Sarrebrück coal deposits, thirteen borings were undertaken, and some account is given of the progress of these works since the spring of 1905, and also of six additional borings put in hand later in the year. The eight borings which have already reached the coal-beds render the existence of coal a matter of certainty in the area round Pont-à-Mousson and Nomeny, covering an extent of ground 15 miles long by about 6 miles in width, or over an area of upwards of 60,000 acres. The coal lies at depths averaging about 875 yards, or from 670 yards in the vicinity of Eply to 1,038 yards near Martincourt. The thicknesses of the various seams encountered are set forth in a table. It is stated generally that the coal seams are comparatively poor throughout the area and will be reached with difficulty. The available thickness of the coal over a large part of the basin will not exceed 10 feet in all, and the total yield might, in accordance with the figures given, reach 1,000 million tons. Some working estimates are given and the positions of the various borings are indicated on a sketch-map. The coal is said to be suitable for coking purposes, but it is somewhat too rich in volatile matters.

G. R. R.

Mechanical and Hand-Firing.

(Engineering and Mining Journal, New York, 18 November, 1905, p. 903.)

The Author, while admitting that mechanical stoking possesses some advantages over hand-firing, believes that it is not of itself more economical. In his opinion any test will result in favour of one or the other according as the men who work the stoker or who fire by hand are the more expert and efficient. In support of his contention he instances trials made with the plant of an electrical company using modern machinery. The boiler was horizontal, return tubular, working at 135 lbs. per square inch pressure. The engines were compound condensing, and induced draught was regularly used. The exhaust steam from the stoker engine and the pumps was used for heating the feed-water. The first trial showed a saving of about 7 per cent. in favour of the mechanical stoker. A second trial with the same plant, under the same conditions, was made within a year from the first. Moreover the test was made by the same men; the only change being in the personnel of the firemen. The result in this case was exactly the reverse of the first, the saving being within a fraction of 7 per cent. in favour of hand-firing. It is stated that neither of the tests was "doctored"; so that the results are not open to question; even the coal was from the same district and had about the same moisture, composition, and calorific value. The efficiency of the firemen was practically the only variable quantity.

G. G. A.

Copper Ore and Gold Quartz at Nietsajoki, Sweden.

E. Svedmark.

(Teknisk Tidskrift, Stockholm, 1905, Chem. and Mining section, p. 124.)

In October 1905 the Author reported upon three copper mines—numbered 3 and 4 and 9—at Nietsajoki in Gellivare parish. In No. 3 the copper ore was met with in 1902, and is chiefly variegated, occurring in the shaft in a quartz gangue about 0·2 metre (8 inches) wide, while the hornblende country to the extent of 1 metre (3¼ feet) on each side is so richly impregnated with variegated copper ore as to be fully worth getting also. In No. 4 the ore is chiefly copper pyrites, and was met with in 1903; by sinking a shaft in 1904 a promising discovery on a large scale was made; down to the bottom the shaft is almost wholly impregnated with ore. In No. 9, where two shafts have been sunk to depths of 3 and 2·5 metres (10 and 8 feet) and a couple of small headings driven, the Author anticipated in 1903 the probability of gold being met with in the quartz veins, because in this set the country rock is in several places of the same kind as in neighbouring sets at Ferrum, where gold has been found in solid rock at four or five points. The quartz veins at Nietsajoki,

both in No. 9 and in No. 4, are of exactly the same kind. A sample taken by Mr. Löfstrand from No. 9, containing 3·71 per cent. copper with as much as 44 grammes of gold per tonne (1·579 oz. per ton), looked like a continuation of the Ferrum discovery. A larger specimen of about 10 kilograms (22 lbs.), taken by the Author for his report and analysed by J. Landin, has been found to contain only 3 grammes of gold per tonne (0·108 oz. per ton), with 16 grammes (0·575 oz. per ton) of silver. The gold is most irregularly disseminated throughout its matrix in minute flakes or spangles. The practical conclusion arrived at is that, although no considerable quantity can be expected at any one point, yet the traces met with are by no means to be despised as a by-product while getting copper ore as the principal mineral.

A. B.

Swedish Iron-Ore Resources.

(Teknisk Tidskrift, Stockholm, 1905, Chem. and Mining section, pp. 73–85.)

On the question of imposing an export duty upon iron ore, a parliamentary committee invited opinions from professional men concerned in the geological exploration of Sweden, as to her iron-ore resources compared with those of other countries. The appeal was responded to in a report drawn up by Professor A. E. Törnebohm, chief of the geological survey; and criticisms of his report were sent in by Professor Hj. Sjögren.

Prefacing the report with a proviso as to the difficulties attending such an investigation, Professor Törnebohm passes under more or less elaborate review the iron and coal districts known in civilized countries, giving the sources from which his information has been derived. His views of the prospects for the future are then summed up in the six following conclusions. (1) As regards North America, Germany, and England—now the three leading countries in iron-making—their iron ores will certainly be worked out within a century or two, and the richer kinds much sooner. (2) Consequent decline or stoppage of the iron industry will occur only in England, because there the coal also will have been all won. (3) In Germany and North America shortage of inland ores will be made up by importing, as long as their reserves of coal will hold out; iron ore travels to coal, not conversely. (4) Beyond the districts already opened, only the province of Shansi in the north of China now presents conditions necessary for the growth of a great iron industry, because only there both coal and iron are found together. But should future progress of science enable iron to be got from its ores without coal, or with only an inconsiderable consumption thereof, the conditions would naturally be so altered as to render nugatory any calculations now attempted. (5) The production of iron seems likely in the course of the present century to depend,

partly upon such of the ores already found in civilized countries as have hitherto been considered too poor or containing too many impurities; and partly upon new discoveries in regions not yet explored geologically. That such discoveries will be made is highly probable, inasmuch as the known extension of the iron-ore fields can hardly be regarded as commensurate with that of the existing civilization. (6) The localities for the future manufacture of iron will be determined by the position of the coal-supply and by the conditions of carriage. By both of these, and by the progress of metallurgy in utilizing the ores, will the future production of iron be governed. Ore enough to meet the world's requirements of iron will probably never be wanting.

Professor Hj. Sjögren, criticizing Professor Törnebohm's report, and estimating along the same lines, substitutes the voluminous data compiled by himself, giving similarly the sources of his information. His own opinions he summarizes in the two following conclusions. (A) The Swedish ores, totalling about 600 million tons, are by no means inexhaustible, nor on the whole more ample than may well be needed for the home industry as time goes on; their export contributes to lowering the nation's position among the countries whose industrial prospects are bright. (B) For a country's financial strength, and for the development of her industries, the possession and treatment of her own iron ores are of such eminent importance as ought to cause a different valuation of Sweden's ore resources from that hitherto obtaining; the output of iron ore in most countries, and especially in Europe, is sure during the present century to reach its meridian and likewise its inevitable decline; and no one having a sense of responsibility can regard without concern the mismanagement which now takes place with Swedish ores.

In a rejoinder to Professor Sjögren's criticism, Professor Törnebohm confines himself to the difference between his own estimate of 1,200 million tons of ore as Sweden's probable resources, and Professor Sjögren's of only half as much. Of this difference nearly 500 million tons are accounted for at the Kiiruna iron-ore mines. The higher estimate has been based upon sections of recent borings, lent by Mr. Hj. Lundbohm; whereas the lower was framed upon only two deep bore-holes put down prior to 1897, which were then believed to indicate that at increasing depth the ore was thinning out in width; no such inference is warranted by the subsequent holes. Both Authors are agreed that Sweden has no ore to waste, for even 1,200 million tons is no great quantity in relation to the world's requirements, and it would be a grievous thing for it to be worked out too soon.

Mr. Hj. Lundbohm, who presented a report in 1897 to the Royal College of Commerce upon the iron-ore resources in Kiirunavaara and Luossavaara, disclaims any conclusions founded thereon in connection with the present comprehensive enquiry, because the investigation he then made was solely with a view to ascertaining whether the district contained ore enough to justify the construction

of a railway from Gellivare to Ofoten.[1] Hence the figures then given were intended to represent a minimum quantity likely to be forthcoming above a certain limit of depth. At that limit it was believed the ore was showing signs of thinning out in width. But this has since been proved to be a mistake. During the 7 years that have since elapsed, a large number of bore-holes have been put down; a score of them aggregated a depth of 2,660 metres (1,450 fathoms). From one there seems every probability that at increasing depth the deposit widens out considerably; while another shows conclusively that it is as wide at 116 metres (63 fathoms) below the level of Luossajärvi lake as it is on the hill top 78 metres (43 fathoms) above that level. The greatest depth to which diamond boring has extended in the ore is 238 metres (130 fathoms) below the surface of the lake. Comprehensive magnetic[2] explorations have also been carried out, which prove that the vast Kiirunavaara iron-ore deposit, unquestionably the largest yet met with in the world, is divided at Luossajärvi lake into two separate beds of 1,300 and 1,100 metres length (1,400 and 1,200 yards), each of great width. From these more recent investigations, coupled with the knowledge acquired in getting and dressing such large quantities of the ore, were compiled the calculations furnished to Professor Törnebohm respecting the iron-ore resources in Kiirunavaara.

A. B.

Welding by the Oxy-Acetylene Jet.

(La Revue Technique, Paris, vol. xxvi. pp. 856-9.)

When, in the ordinary oxy-hydrogen jet, the hydrogen is replaced by acetylene, a temperature of about 4,000° C. is obtained, as compared with little more than half that intensity when hydrogen is used. This increase in temperature might be expected from the fact that, whereas a cubic metre of hydrogen is only capable of giving out 3,100 calories, the same volume of acetylene can give out 14,700. In the oxy-acetylene jet the correct proportion of the two gases can be regulated to a nicety, as the character of the flame alters completely when either is in excess. The flame also possesses a very clearly-defined reducing zone, the presence of which for the purpose of welding is of the greatest importance. Taking the cost of oxygen as 5 francs per cubic metre (1·35*d.* per cubic foot), that of hydrogen at 4 francs per cubic metre (1·08*d.* per cubic foot), and that of acetylene at 1·5 franc per cubic metre (0·405*d.* per cubic foot), the cost per calorie works out at 0·00338 franc (0·03*d.*) in the case of hydrogen as against 0·00047 franc (0·005*d.*) in the case of acetylene.

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[1] Minutes of Proceedings Inst. C.E., vol. clii, p. 346; vol. clvi, pp. 449, 450, 451, 453; vol. clxi, p. 376.

[2] *Ibid*, vol. xcviii, p. 473

The apparatus is provided with interchangeable nozzles to suit the different thicknesses of plates operated on; the smallest of these allows 55 litres (1·92 cubic foot) of mixture to pass per hour, while the largest passes forty times this quantity. In using the apparatus the pieces to be burnt together are clamped in contact with each other and the seam to be welded is made to assume a horizontal position, oxygen is then admitted to the jet until the flame is colourless at its extremity while strongly luminous and of a bluish tinge near the orifice. With a view of showing the capabilities of the apparatus, a series of tests were made in the presence of engineers appointed by the Bureau Veritas. One of the most severe of these was the welding of two similar angle-bars, the ends of which had been cut to a mitre, the corner piece thus made being tested as to soundness by being flattened out. Similar bars were welded in the ordinary way, and those burnt together by the jet were found to be the stronger. Comparing the cost of riveting and of burning together a long seam, the jet works out the cheaper when the thickness of the pieces is under 8 millimetres (0·31 inch).

I. C. B.

Heating Effects produced by Röntgen Rays in Metals, and their Relation to Changes in the Atom. H. A. Bumstead.

(American Journal of Science, New Haven, Connecticut, 1906, vol. xxi. pp. 1-24.)

The Author gives an account of his investigation of the heating effects produced in lead and zinc by the absorption of Röntgen rays. The experiments show that when Röntgen rays are equally absorbed in these two metals, approximately twice as much heat is produced in the lead as in the zinc, and the Author can only account for this effect by the hypothesis that by these rays the atoms of certain elements may be artificially broken up, and that the energy thus liberated forms a part, and perhaps the greater part, of the energy which appears when the rays are absorbed by matter.

W. C. H.

Boiling and Distillation of Metals of the Iron Group.

H. Moissan.

(Comptes Rendus de l'Académie des Sciences, Paris, 1906, vol. cxlii. pp. 425-30.)

In this Paper the Author describes the results of experiments made on the boiling and distillation of iron and other metals by means of the electric furnace. He finds that the metals of the iron group have very different boiling-points. Manganese is the most volatile of all. After it comes nickel, with which metal boiling takes place quite quietly; then chromium, which distils steadily

under the action of a current of 500 amperes at 110 volts. The boiling of iron is much more difficult, and is preceded by a violent escape of the gases which this metal so easily dissolves, but by employing stronger currents, and after the first effervescence has subsided, the iron may be brought to a steady state of boiling. With a current of 1,000 amperes at 110 volts the Author distilled 400 grammes (0·88 lb.) of iron in 20 minutes. Uranium has a higher boiling-point than iron, and its distillation is only produced with a current of 700 amperes at 110 volts after 5 minutes heating in the furnace. Molybdenum and tungsten, on the other hand, are much more difficult to bring to a state of boiling, and with a current of 700 amperes at 110 volts the Author has only been able to arrive at steady boiling of the latter metal after 20 minutes. In all these experiments the crystalline dust obtained by condensation of the metallic vapour possesses the same chemical properties as the metal reduced to fine powder.

W. C. H.

Nickel-Manganese Steels. L. Guillet.

(Bulletin de la Société d'Encouragement pour l'Industrie Nationale, Revue de Métallurgie, Paris, 30 November, 1905, pp. 825–41.)

This investigation, it is stated, is undertaken as representing the first group of quaternary steels, which the Author proposes to study in detail. Mr. Hadfield has already examined certain of these alloys, and he has ascertained that a sample containing 0·6 per cent. of carbon, 14·5 per cent. of nickel and 5·04 per cent. of manganese is not brittle, whereas with the same proportions of nickel or of manganese alone a steel would be rendered very ill-adapted to resist an impact. Mr. Dumas has also noticed some interesting examples of such alloys in his work on nickel steel. By reference to a triangular diagram, in which the proportions of carbon, nickel and manganese are set off on lines, drawn through the angles to the centre, the Author discusses the properties of the various types of steel depending on the relative quantities of nickel or manganese they may contain. In order to verify the results theoretically ascertained, a number of samples of steel were prepared by the Imphy Steel Company. The first series was devised in order to study the influence of manganese on nickel steels with high and low carbon contents. In the case of one set of samples 5·7 per cent. of manganese was employed and 15 per cent. of manganese for the other set. Complete analyses of each are shown in tables. From the indications deduced from these experiments a third series of samples were made at Imphy, of which the analyses are appended and, by reference to micrographic diagrams, the appearances of the different characteristics shown by each of the various alloys are explained. The mechanical properties are then investigated by means of tensile tests, tempering, reheating and chilling, and the behaviour of the steel under each of these treat-

ments is examined, liquid air being used for the cold tests. In conclusion the Author shows that nickel-manganese steels are of three types—pearlitic, martensitic, and γ iron. In the martensitic steels it is necessary to distinguish those which consist of ordinary martensite and show spiculæ capable of being readily coloured. It is suggested that those needles which disappear when heated up to 850° C. may be sorbitic in character. It is shown that these steels follow the laws represented by a very simple diagram and partake to some extent of the qualities of ordinary alloys either of nickel or manganese. G. R. R.

Nickel-Vanadium Steels. L. Guillet.

(Bulletin de la Société d'Encouragement pour l'Industrie Nationale, Revue de Métallurgie, Paris, 31 December, 1905, pp. 870–81.)

The chief characteristics of nickel steel and of vanadium steel are first briefly reviewed, and the Author states that, in order to study the properties of nickel-vanadium steels, a series of alloys were prepared for him at the Imphy steelworks, the one set with low carbon, amounting to about 0·2 per cent., and the others with carbon averaging about 0·8 per cent. For each of these series a pearlitic, martensitic, nickel steel containing iron was employed, to which various amounts of vanadium were added ranging from 0·2 to 7 per cent. Full analyses of the different samples are given in a table, and the Author explains the micro-structure by reference to enlarged photographs. It is pointed out that by this means he has demonstrated the existence of six classes of nickel-vanadium steels: (1) Pearlitic steels; (2) carbide and ferrite steels; (3) martensitic steels; (4) steels with martensite and carbide; (5) steels with γ iron; and (6) steels with γ iron and carbide. The only feature of interest to be recorded is that the presence of vanadium would seem to lower to some extent the points of transition from the one group to the other of nickel steels. The mechanical properties of these steels are shown in a table, and certain peculiarities are explained by reference to micro-photographs. As a general result it would seem that the addition of vanadium to nickel steels is only of interest in the case of those of the pearlitic type. The behaviour of these steels, when subjected to various kinds of treatment, is then discussed—tempering, re-heating, etc. In conclusion it is pointed out that, in the case of steels of normal type, the addition of vanadium tends to increase the tensile strength and the elastic limit of pearlitic steels more rapidly than any other element with the exception of carbon. The maximum effect is produced by the employment of from 0·7 to 1 per cent. of vanadium. In the case of tempered pearlitic steels the action of vanadium in homœopathic doses is quite extraordinary in its influence on the tensile strength as compared with that of the normal steel. G. R. R.

Influence of Aluminium and Tungsten in Steel on Corrosion in Dilute Sulphuric Acid. O. Bauer.

(Mitteilungen aus dem Koeniglichen Materialpruefungsamt, Gross Lichterfelde West, 1905, vol. xxiii. p. 292.)

At the request of Mr. Parfitt, of Cardiff, two samples of steel alloy were prepared by different methods, but both were intended to contain 0·5 per cent. aluminium and 0·3 per cent. tungsten. The samples were cast, but not hammered, and cut into slices, having exposed surfaces of 4 to 5¼ square inches. They were placed in 1 per cent. solutions of sulphuric acid, which was occasionally renewed, and the corrosion was determined by weighing. The experiments were continued for 288 hours, during which time the losses expressed in inches were as follows:—

Sample I.—0·0056, 0·0048, 0·0051, 0·0051 inch.
Sample II.—0·0063, 0·0060, 0·0065, 0·0054 inch.

C. E. S.

Application of Electricity on Board Warships.

P. Kergarouët.

(Éclairage Electrique, Paris, 1905, vol. xlv. pp. 377–87, *et seq.*)

In this Paper the Author gives a detailed description of the best practice in electrical installations on board modern warships. Special conditions have here to be considered in deciding on design and equipment; in particular, economy in cost of installation and in working expenditure, which are the determining factors in industrial installations, are here of less importance than the assurance of maximum of safety and reliability in operation at all times and under all conditions. The Author therefore discusses the problem from this point of view, and in dealing with each class of apparatus or machinery, dwells only on the details relating to the special conditions which have to be met. The Author examines in this manner the engines and dynamos, with special reference to their output, speed-regulation, position in the ship, and grouping; the method of distribution employed, size of conductors, insulation, junction-boxes and switches; and finally the mechanisms and apparatus to which current is supplied—electric motors, lamps and search-lights, and signalling apparatus.

W. C. H.

Ridding Ships of Rats. KLAS SONDÉN.

(Teknisk Tidskrift, Stockholm, 1905, General section, pp. 315-7.)

Besides being liable to pestilence themselves, rats are the means of bringing it from foreign ports in ships which they infest. Fumigation with sulphur damages certain cargoes, without proving a sure mode of killing the rats. Carbonic acid gas has been tried; but, though a candle goes out in air containing 12 per cent., the rats are not killed even when the percentage is raised to 30, which is too high to be practicable for use. Independent attempts have been made in England and in Germany to employ producer-gas, on account of its carbonic oxide being such a strong deadly poison for all warm-blooded animals. As Engineer to the Swedish Public Health Department in Stockholm, the Author was deputed to investigate the mode in which this plan has been tried at Hamburg with apparatus devised by the port medical officer, Dr. Nocht, and his assistant, Mr. G. Giemsa.

A gas-producer, steam-boiler and engine, scrubber, and centrifugal fan and pump, are arranged together on board a lighter, which is moored alongside the infested ship. The producer, lined with refractory brick, is charged with coke through a funnel at top that can be closed tight. Clinkering is done through a door alongside the grate; and the height of the coke in the producer is observed through a sight-hole. Air is blown in at bottom by the fan. The mixed carbonic acid and carbonic oxide gases, after passing through the boiler and raising steam therein, are further cooled and washed in the scrubber, which is supplied with water by the pump. Thence they are driven into the ship through a main hose of 100 millimetres bore (4 inches), having four branches of 75 millimetres (3 inches). Two such branches suffice for a production of about 400 cubic metres of gas per hour (say 14,000 cubic feet). The gas driven into the ship is constantly analysed: its percentage of carbonic oxide ranges from 3·3 to 6·6; carbonic acid 19 to 17; nitrogen 77·7 to 76·4 per cent.; specific gravity of the whole 1·085 (air = 1·000). For starting the apparatus, the producer is worked at first as an ordinary boiler-furnace by the aid of a chimney-draught, until steam enough is raised to drive the fan; then the chimney damper is closed, and the gases are driven through the scrubber. After the work has been done, the producer and scrubber are shut off by a valve, and the ship is aired by the fan, either by exhausting the gas from her by suction, or by blowing fresh air into her under pressure. A sketch is given of the general arrangement, showing also the four different stages of working.

When the volume of gas driven into the ship is half her cubic measurement, and is left in for a couple of hours with all air-holes stopped, all the rats are killed. With a larger volume of gas, a shorter time suffices. For gauging whether the ship was filled with gas enough, rats in cages were lowered into it to see how quickly they were killed. After she had been aired, similar tests were

made; and no one went below, until caged rats had been kept down for a couple of hours without showing any signs of poisoning. An example is given of a steamer of 8,768 cubic metres capacity (309,656 cubic feet), with a cargo of wool, hides, and linseed from La Plata. Rats and mice were killed in her to the number of 285, of which five were pestilential. The gas had penetrated the remotest recesses of the chinks between the sacks in the hold. On viler vermin it seems hardly likely that the gas has any effect.

For the large traffic and big ships at Hamburg this first apparatus is too small. It is intended to construct a new lighter, furnished also with baths and disinfecting arrangements for the ships' crews, at an estimated cost of from 113,000 to 116,000 marks (£5,650 to £5,800). The staff required is so small, that the chief item of working expenses is the coke; if this costs 1 krona per hectolitre = 40 kilograms (say 4·85*d.* per bushel or 1*s.* 5*d.* per cwt.), the gas will cost a little more than 3 kronor per 1,000 cubic metres (say 1·14*d.* per 1,000 cubic feet).

A. B.

New Water-Supply for Malvern.

(The Engineer, London, 6 April, 1906, p. 356.)

This supply is obtained from Bromsberrow, where two boreholes have been sunk to a depth of 200 feet into the new red sandstone. These boreholes are 40 feet apart, and are lined to a depth of 60 feet with 15½-inch-diameter steel tubes.

The pumping plant is provided with steam by two Lancashire boilers 28 feet long and 7 feet in diameter, working at a pressure of 130 lbs. per square inch, one boiler being always in reserve. The engines driving the pumps are of the compound coupled type, developing 120 B.HP. The water level in the boreholes is 31 feet below the surface, and a deep-well pump has been fixed in each borehole, designed to lift 900,000 gallons of water per day, delivering into surface tanks, each capable of holding 8,000 gallons of water. Two surface pumps force the water to the storage reservoir 5¾ miles distant, the quantity being measured by a Venturi meter. The pipes are 10 inches in diameter; the pipe-line is divided into sixteen sections by stop valves; automatic back-pressure valves are also provided at various points to relieve the pressure when the engines are standing, and to prevent more than the section between two valves emptying in case of a burst.

A. W. B.

Completion of the New Croton Dam.

(Engineering Record, New York, 6 January, 1906, pp. 7-9.)

The practical completion of this enormous work to supply New York with water is recorded. It was begun in 1892, and will cost, with all accessories, but exclusive of land purchase, about £1,600,000. The dam is exactly 300 feet high from the lowest point in the foundations to the crest, and is thus the highest dam in the world. It forms a reservoir 18 miles long and impounds about 32,000,000,000 gallons. It is of solid granite ashlar and rubble, except that the south extension is backed with cyclopean concrete. There is a spillway 1,000 feet long at right angles to the axis of the dam and 16 feet below its crest.

There is about 855,000 cubic yards of masonry in the structure. The spillway is crossed by a steel arch of 200 feet span carrying the 20-feet roadway, which occupies the crest of the dam, the bridge having been described in the *Engineering Record* of the 21st October, 1905.

In connection with the necessary diversion of the river, earth was excavated by hand and by steam shovels, and removed to spoil-banks below dam, by teams, by locomotives, by an inclined plane and hoisting engine serving depths of over 70 feet, and finally by three cableways of about 2,000 feet long parallel with the axis of the dam, and nearly 300 feet above the lowest excavation.

The masonry was laid by the aid of the cableways and of about thirty large stiff-leg steam derricks, which were carried to the top of the masonry as it progressed. The south end of the dam was originally designed and partly constructed with earth embankment and a core wall, but it was decided to alter it to solid masonry, and the foundations were carried down deeper.

The face of the dam is built with granite ashlar in courses 20 to 30 inches in depth, and the rubble backing was laid with special care. Large stones were set on beds of concrete and mortar, in which they were fitted by bedding and then lifting them again, with derricks, several times, care being taken to fill in plenty of mortar over all irregularities. The 85,000 cubic yards of concrete backing in the dam extension contain large stones lowered into fresh concrete, special precautions being taken to set them with convex faces downward, so as to avoid air cavities.

A full description is given of the spillway, which is formed of steps of granite blocks, and held together by heavy anchor-bolts.

C. O. B.

Sheet-Steel Piles in Reservoir Construction.

(Engineering Record, New York, 18 November, 1905, pp. 571-2.)

A peculiarity in the foundation work of a dam is the chief subject of an interesting article. The dam is an earthen one, with a maximum height of 35 feet, 1,600 feet long at its crest, 24 feet wide at

top, and 2 to 1 slopes, and it has a concrete core. The reservoir, which is at New Milford, and has a capacity of 870,000,000 gallons, supplies water to several towns in North-Eastern New Jersey. Owing to the pervious nature of the subsoil, considerable leakage and danger to the structure would have occurred if the concrete had not been carried down to the underlying rock, or some equivalent expedient adopted. As such a deep core wall would have been too expensive, it was decided to strip the top soil from the dam site to a depth of 12 to 18 inches, and drive interlocking steel sheet-piling down approximately to the rock; to imbed the top of the piling in the base of a concrete core-wall, and then to build up around the latter the earthen dam. The saving by this was estimated to amount to £40,000. The piling was delivered in 30-foot lengths, and driven to a depth of 35 to 50 feet, the lengths being spliced together on the ground where necessary. Large boulders, which caused much trouble, were frequently encountered. An ordinary land pile-driver, with a 2,000-lb. hammer, was employed; but in order to get the piles around the boulders, two water-jets had to be used. A 1,000,000-gallon Worthington pump was installed on the river-bank, and a 6-inch main laid from it the entire length of the dam. Connections for 2-inch hose-pipe were made on this main, and the hose connected with 1·5-inch pipe, which was sunk in the usual manner of a water-jet, one on each side of the pile as it was driven. In this manner it was possible to wash holes around boulders, so that they could be shoved to one side by the piling. When the boulders were too large for this treatment, the piling was deflected around the boulder. No attempt was made to cut off the piling at the top, so that it projected irregularly into the concrete above.

Dependence was placed entirely on the core-wall and sheet-piling to prevent any flow of water through the material under the dam. Any water that might pass under the piling, or through the joints in it, was held to be subject to such a loss of head due to the excessive friction encountered in filtering through the soil for such long distances, and on account of silting, that the leakage would be slight. Gaugings since have shown it to amount to less than 500,000 gallons a day. Observations, taken after the reservoir was filled, showed the slope of the underground water to be about 5 feet in 1,000 feet; the slope produced by driving the sheet-piling being changed to about 175 feet in 1,000 feet.

C. O. B.

Reinforced-Concrete Water-Tower at Bordentown, New Jersey.

(Engineering Record, New York, 13 January, 1906, pp. 39–41.)

The waterworks at Bordentown include a steel tank of 200,000 gallons capacity, 30 feet in diameter and 40 feet high, placed on a reinforced-concrete tower, which has some interesting features. It

is 100 feet high from the top of its footing course to the floor on which the tank stands, and is formed by eight vertical columns, spaced equally round the circumference of a 26-foot 6-inch circle, and a central hollow concrete cylinder, 8 feet in diameter, the latter being connected with the columns by the floors of two balconies dividing the tower into three stages 33 feet 4 inches high each, and the top 36 feet 6 inches in diameter and 1 foot 4 inches thick carrying the tank. The concrete footing on which the structure rests is a sixteen-sided polygon in plan, 6 feet thick, 38 feet in diameter at bottom and 32 feet at top. The columns are 3 feet 6 inches square up to the first balcony, and 3 feet and 2 feet 6 inches square in the second and third stages, the inner sides being in the same plane throughout, and the reduction made by stepping the other three sides. The central cylinder is provided to reduce the load on the columns, and to give resistance to flexure due to wind-pressure. It has a thickness in the three stages of 18 inches, 15 inches, and 12 inches respectively, and was built, together with the rest of the structure, monolithically. The supply and overflow pipes are within the cylinder.

With regard to the reinforcement, each column has four vertical round rods, one near each corner and varying from $1\frac{7}{8}$ inch to $1\frac{3}{8}$ inch in diameter, according to height. Horizontal tie-rods, $\frac{1}{4}$ inch in diameter, are placed on 12-inch centres around the vertical rods. The latter were provided chiefly to take compression and are embedded 12 inches in the concrete footing. Their bottoms rest, by means of 2-inch pipe sleeves, on special cast-iron bases 12 inches in diameter.

The cylinder is reinforced with 1-inch and $\frac{7}{8}$-inch vertical rods in first and in second and third stages respectively, and horizontal $\frac{1}{4}$-inch tie-rods are placed on 12-inch centres around the vertical rods. The balcony floors are reinforced with $\frac{1}{2}$-inch corrugated radial bars, providing also for the cantilevering of the top one. A reinforced-concrete spiral stairway is provided through the balconies.

The concrete in the footing was 1, 3, 5, and in the tower 1, 2, 4, and Alpha Portland cement was used. The tenders were alternative for steel and reinforced-concrete construction, and out of eleven, four favoured the latter, six favoured the former, and in one the rate was the same for each. The superior appearance of the type chosen decided the matter, the cost being about £2,100.

C. O. B.

Experiments with the Pitot Tube in Measuring Velocities of Gases in Pipes. R. BURNHAM.

(Engineering News, New York, 21 December, 1905, pp. 660-2.)

The experiments were carried out for the purpose of rating Pitot tubes for measuring the velocity of flow of gases. The form of tube used consisted of two brass tubes one within the other, the inner one,

$\frac{3}{16}$ inch outside diameter and $\frac{1}{32}$ inch thick, forming the velocity tube; the outer or pressure tube was made of $\frac{3}{8}$-inch tubing, $\frac{1}{32}$ inch thick and provided with a slit $1\frac{1}{4}$ inch by $\frac{1}{16}$ inch on the under side of the horizontal portion for transmitting the pressure. It was found that the shape, within wide limits, of the mouth of the tube facing the stream, makes no appreciable difference in its indications, a continuation of the horizontal part of the outer tube beyond the packing between the two answering quite well. The difference of pressures indicated in the two tubes was transmitted through india-rubber pipes to opposite sides of a sensitive manometer. The tubes could be moved through a stuffing-box so that the tip could take up any position across the pipe diameter.

Two illuminating gas-holders, having capacities of 900,000 and 2,000,000 cubic feet respectively, communicating with each other through a 24-inch diameter cast-iron main, free from bends or other obstructions for a length of 150 feet, gave facilities for carrying on the experiments. A Root exhauster, capable of delivering 17,000 cubic feet per minute, served to draw the gas from the smaller and deliver it into the larger holder, the velocity of flow being controlled by the speed of the exhauster. The normal pressure in the large holder was several inches of water in excess of that in the smaller, which permitted the gas to be returned at any desired velocity by manipulating a by-pass valve. Two Pitot tubes were inserted near the middle of the straight portion and a thermometer midway between the two; one measured the velocity at the centre continuously and the other was moved across the diameter of the pipe. The true mean velocity of the gas was obtained by recording the rise or fall of the smaller holder at frequent intervals during the run and computing the volume displaced in cubic feet per second from the known dimensions of the holder. This, reduced to the temperature and pressure recorded at the Pitot tubes and divided by the cross-sectional area of the pipe, gives the mean velocity. From these results the value of the coefficient K, in the formula $v = K\sqrt{2gh}$ for velocities ranging from 35 to 65 feet per second, was found to be unity.

A. W. B.

Indirect Illumination of Schoolrooms and Drawing Offices.

(Zeitschrift für Architektur und Ingenieurwesen, Weisbaden, 1905, part vi. pp. 535–8.)

A Committee of the German Society of Gas- and Water-Engineers was appointed at the forty-third annual meeting of that Society to make comparative experiments as between gas and the electric arc in the illumination of schools and drawing-offices by means of fully diffused and semi-diffused light.

The Committee now report that, starting with the assumption that in drawing-offices an intensity of 80 lux (on a white surface) is required at the drawing-board and 25 lux in schools and lecture

theatres, they found that their requirements were not only satisfied, but could be exceeded by both gas and electricity in the installations tested.

In all the experiments, the distribution of light was more regular with gas than with electricity, on account of the larger number of lamps used. In lecture theatres the brightness in both cases was disagreeably noticeable on the higher benches. In the drawing-office test the inequality of distribution was insignificant with the carbons in their normal position, but with reversed carbons there was a greater difference, due to the small diameter of the circle of illumination thrown on the ceiling by the lower carbon.

As regards fluctuation of intensity, in neither case was any sudden change photometrically measurable. A slight flicker was present in three-series lamps, on which account these are not to be recommended for schools or lecture theatres without a resistance; on the other hand, gradual changes in intensity were distinctly measurable, but being transitory were of no practical significance.

The reduction of illuminating power with time for mantles was small within the limits of practical use, and amounted to not more than 5·9 per cent. after 300 hours, and 13·5 per cent. after 600 hours.

The formation of shadows from either source was only marked in semi-diffused light. The composition of the air in unoccupied rooms did not change much with electric lighting; but with gas, even with the latest arrangements, except where ventilation was provided, there was a marked increase in the percentage of carbonic acid in a short time, and the temperature was considerably greater for gas than for electricity; on the other hand, it was found that with the crudest ventilation, such as openings near the ceiling, these faults were modified to a quite unexpected extent, and in occupied rooms the carbonic acid was found to be actually less for gas at the end of the experiment, which is attributed to the increased circulation due to higher temperature.

As regards cost, at 2½*d.* per cubic metre for gas and ¾*d.* per kilowatt hour, in the case of semi-diffused lighting and moderate intensity (25 lux), for two-series lamps the cost was roughly three times, and with three-series lamps twice as much as for gas. With diffused lighting and greater intensity (80 lux) the cost of illumination was least with pressure-gas, the cost of the arc with normal carbons being 2¼ times greater, and with reversed carbons 1½ time greater, the cost of ordinary gas being about equal to the arc lamp with reversed carbons.

Considering all incidental expenses, in the present case it was found that the comparative cost was as follows:—Semi-diffused light (25 lux) 1; electric arc (two lamps in series) 2·8; ditto with three lamps in series 2·0; completely diffused light (80 lux):—pressure gas 1·0 to 1·1; ordinary gas mantles 1·2; electric arc with normal carbons 1·6; with reversed carbons 1·1.

E. H. S.

Efficiency Meter for Incandescent Lamps. E. P. Hyde and H. B. Brooks.

(Electrical World, New York, 1905, vol. xlvi. pp. 942–5.)

The life of an incandescent lamp depends on its efficiency, that is, on the ratio "watts per candle." In practical photometric tests this ratio has to be found by calculation, but the Authors of this Paper describe an instrument they have designed and constructed for use in the American Bureau of Standards, which gives the efficiency by direct readings on the photometer. The principle of the method employed is the automatic insertion of a definitely-graded extra resistance into the pressure circuit of the wattmeter by a pair of sliding contacts attached to the photometer carriage. The Authors work out the law which this resistance must follow to give the desired direct readings of efficiency, and show how obedience to the law may be secured by winding the resistance on a frame of definite geometrical form. The actual reading of the instrument is the product of the efficiency by a constant, and by giving this constant the value 10, the true watts per candle are at once obtained by shifting the decimal point, and at the same time the instrument reading comes on a good point of the scale. The variable resistance is wound on two insulating blocks of the proper form, placed side by side and identical in every respect. A strip of the insulation is removed along the top of each block and a brush, carried by the photometer carriage, spans the gap between the two coils and rests on the exposed parts of the resistance winding. An illustration is given in the Paper, showing the blocks in position on the photometer. The experimental results show that the instrument is remarkably accurate, giving for 16 candle-power lamps ranging in efficiency from 2·4 to 4·8 watts per candle true results to within less than 1 per cent. The efficiency meter may also be used for determining directly the voltage at which a lamp has a given efficiency.

W. C. H.

Calculation of Dynamo Sparking Constants. A. Press.

(Electrical World, New York, 1906, vol. xlvii. pp. 525–7.)

The object of this Paper is to explain a short and simple method of calculation of the factors which have to be considered by the designer of electric generators in dealing with sparking at the commutator. Three things have to be considered:—(1) The magnetic momentum of the short-circuited coil; (2) The voltage generated in the coil due to the coil cutting lines of force; and (3) The voltage drop in the short-circuited coil spanned by the tip of the brush at the last instant. The Author deals with these three matters

and develops formulas for the sparking constants, which are easy to compute and have been checked in practice by reference to a large number of machines. In every case the qualities of the machine with regard to sparking have been determinable from the magnitude of the sparking constants.

W. C. H.

Commutation in Single-Phase Motors at Starting. M. LATOUR.

(Electrical World, New York, 1906, vol. xlvii. pp. 522–3.)

In this Paper the Author studies the commutation of single-phase motors at starting, where resistances are used in series with the coil under commutation. He treats the problem by three different methods, each of which is in itself incomplete, and then finds certain relations existing between the results thus separately obtained, and by combining them is enabled to arrive at the complete solution. Acting on the hypothesis that the heat produced at the brush should be a minimum, he investigates (1) the average quantity of heat produced per second per unit area under the brush; (2) the average quantity of heat produced per second per unit area at the edge of the brush; and (3) the voltage between brush and bar at the moment when the brush leaves the bar. The investigation is mathematical and leads to various equations of a somewhat complex character.

W. C. H.

Theory of Commutation and Exact Diagrams of Single-Phase Motors. F. NIETHAMMER.

(Eclairage Electrique, Paris, 1906, vol. xlvi. pp. 81–102, 130–44, and 161–70.)

The Author discusses at length the theory of the series, repulsion, and compensated types of single-phase motor, and develops exact diagrams for them. After dealing with the general diagram the Author discusses his subjects under the following heads: the couple; commutation and production of sparks; power-factor; starting; speed-regulation; braking; and motor losses. The Author points out the difficulties which have to be dealt with in the practical design of these types of motors, and explains how these have been overcome in the machines now on the market. He also indicates the relative advantages and disadvantages of the different types.

W. C. H.

Toronto Electrical Sub-Station.

(Electrical World, New York, 1906, vol. xlvii. pp. 559–63.)

This Paper contains a description of the Toronto main sub-station, the largest sub-station yet erected as regards capacity. When fully equipped it will receive 30,000 kilowatts transmitted from Niagara, 75 miles distant, at a pressure of 60,000 volts, and will distribute this energy at 12,000 volts for the electric light and power system of Toronto, and for the tramways in and around that city. Apart from its huge capacity, the station is noteworthy on account of the great precautions taken to isolate separate circuits and to protect all the high-tension lines. There are four separate transmission circuits to the station, each at 60,000 volts, and of 7,500 kilowatts capacity, and each is treated as an independent unit, and provided at the sub-station with its own bank of three transformers, each of 2,400 kilowatts output, while a fifth similar bank of three transformers is installed as a reserve. The arrangement of switches and bus-bars is such that each transmission circuit may be directly connected with any of the five groups of transformers alone, or may be connected with two or more groups of transformers in parallel. On entering the sub-station each conductor of a transmission circuit is connected to a lightning arrester containing 240 air-gaps between brass cylinders, and sixty carborundum rods in series between the conductor and the earth. The transmission and distribution is by three-phase currents, and each group of three transformers is connected in delta at both its primary and its secondary windings, so that each transformer has the full time-pressure at its high-tension coils. The high-tension switches on the conductors of the three-phase transmission lines are of special design and construction, and each is located in a separate brick and stone compartment, 4·5 feet long, 3 feet broad, and 7·75 feet high. These compartments are separated from each other by fire-brick walls 8½ inches thick. Each switch is immersed in heavy petroleum contained in a pine-wood bath, and all operations throughout the station are made by remote control. The transformers are all oil-insulated and water-cooled, and each is located in a separate chamber, having brick walls and a steel door. The fifteen chambers are arranged on the same level in a single row. On the secondary circuits of the transformers there are thirty-one motor-operated 12,000-volt oil-switches for three-phase circuits, twenty-five of which are rated at 300 amperes, and six at 500 amperes. Each switch is located in three brick and stone cells, similar to those above described. The total throw in these switches is 40 inches; in the 60,000-volt switches it is 66 inches. The Paper gives details of the design and construction of all switches, the arrangement of bus-bars and connections with them, and the cellular construction adopted for the protection of all the lines. The building measures 61 feet by 202 feet, and contains two storeys and a basement. Practically no wood is to be found in any of its parts. W. C. H.

Effect of Weather on Wireless-Telegraphy Signals.
L. D. WILDMAN.

(Electrical World, New York, 1906, vol. xlvii. pp. 320–2.)

The object of this Paper is to show that in a formula intended to indicate the limit of distance to which signals may be sent from any wireless-telegraphy station, the weather conditions, and especially the factors of moisture and wind-velocity must be taken into account. For a whole year the Author conducted experiments in Alaska, between the army stations at Fort St. Michael and Safety Harbour, recording throughout that time all the atmospheric variables which appeared to have any effect on the signals. When the curve showing wind-velocity is plotted on the same chart as that showing the relative weakness of the signals received at the receiving station, it is seen that there is a general agreement between the two curves, the strength of the signals decreasing with the wind-velocity to a very marked degree. The absorption of energy from the aerial wire at the sending station, due to particles of water, vapour, ice or snow, driven by the wind against the wires charged to a high potential, must therefore be taken into account. No separate investigation of the effect on the receiving antenna has, as yet, been made. W. C. H.

Lightning Flash through Telephone Wire. AUG. WIJKANDER.

(Teknisk Tidskrift, Stockholm, 1905, General section, pp. 299–300.)

In the summer of 1905 a flash of lightning struck the telephone wire of a villa at Lerum railway station, Sweden. The central telephone station is situated near the railway station, low down in a valley, through which runs the river Säfve. Thence is a steep rise of about a third of a mile up to the villa, which stands nearly at the top of that side of the valley, at a considerable height above the central station. The double-wire conductor was attached to the chimney, and was led down through the upper storey and staircase to the telephone below. There was no lightning conductor, nor was the telephone earthed. Running over a plinth on the staircase, it was here left bare; and the lady of the house, happening to be going upstairs and passing within a few inches of that part at the instant of the flash, was killed on the spot. After describing in detail the course of the current and the trifling damage done in the house itself, the Author urges the importance of earthing the telephone; and protests against the common practice of carrying the wires along cornices and elsewhere, in such a way as to be out of sight, and inconvenient to get at for repair. He recommends that they should not be brought down within easy reach of the inmates of a house, except at the telephone itself; and that, wherever they have been improperly fixed, they should promptly be replaced.

A. B.

Power-Plant Economies. HENRY G. STOTT.

(Engineering Record, New York, 3 February, 1906, pp. 131-6.)

A long informing Paper with numerous tables and diagrams. After a sketch of power-plant development in the last 20 years, the results of the introduction of the steam-turbine and its stimulating effect on power-plant design are discussed. Reciprocating steam-engines, and internal-combustion engines are shown to be on their defence, and to be making important improvements in answer to the challenge. From these facts the Author anticipates very large developments within the next 10 years. To prove his case, which is the proposal to use the steam-turbine and the gas-engine in one plant, three valuable tables are given. The first gives the average losses in B.Th.U. under seventeen heads in the conversion of 1 lb. of coal into electricity in one of the most efficient plants in existence. This is followed by explanatory paragraphs on each head, illustrated by nine diagrams. The second table gives the analysis of the average losses in the conversion of 1 lb. of coal, containing 12,500 B.Th.U., into electricity, according to what is believed to be the best results obtained in Europe and the United States, up to date in the formation and utilization of producer gas. The defects of the gas-engine are commented on, and the combination of the two systems explained, as giving improved efficiency, and removing the only valid objection to the gas-engine. It is stated that a steam-turbine unit can be easily designed to take care of 100 per cent. overload for a few seconds; and as the load fluctuations in any plant will probably not average more than 25 per cent. with a maximum of 50 per cent. for a few seconds, it would seem that if a plant were designed to operate normally with 50 per cent. of its capacity in gas-engines, and 50 per cent. in steam-turbines, any fluctuations of load likely to arise in practice would be taken care of. The Author then goes on to explain how, by utilizing the waste heat in the gas-engines for the purpose of assisting to make steam for the turbines, about 37 per cent. of the total heat lost in the gas-engine could be saved. The third table gives the distribution of the maintenance and operation charges per kilowatt-hour, in a power-plant partly from actual data and partly estimated, under five different systems, viz.:—(1) Reciprocating engines; (2) steam-turbines; (3) reciprocating engines and steam-turbines; (4) gas-engines; and (5) gas-engines and steam-turbines; the results being greatly in favour of the last. Taking the total of the charges for No. 1 at 100, those of the others are 79·64, 75·72, 50·67, and 46·32 respectively.

The Author summarizes his conclusions as follows:—

(1) The present type of steam power-plant can be improved in efficiency about 25 per cent. by the use of more scientific methods in the boiler-room, by the use of superheat, and by running the present types of reciprocating engines high pressure, and adding a steam-turbine in the exhaust between the engine and the condenser. At the same time, the output of the plant can be increased to double

its present capacity, at a comparatively small cost for turbines and boilers.

(2) The steam-turbine plant has an inherent economy 20 per cent. better than the best type of reciprocating engine plant, not so much due to its higher thermal efficiency, as to a variety of causes shown in the third table.

(3) An internal-combustion engine plant in combination with a steam-turbine plant, offers the most attractive proposition for efficiency and reliability to-day, with the possibility of producing the kilowatt-hour for less than one-half its present cost.

C. O. B.

Hydro-Electric Power Scheme at Chittenden, U.S.A.

(Electrical World, New York, 1905, vol. xlvi. pp. 939–42.)

The most interesting feature of the power scheme described in this Paper is found in the fact that the water-power is developed by storing in reservoirs the ordinary drainage of a total area of 24 square miles. The Chittenden reservoir, lying about 1,600 feet above sea-level, and covering 800 acres, receives the drainage of 15 square miles and stores 435,000,000 cubic feet of water. Five miles lower down the same stream another dam creates a reservoir of 320 acres, which receives the drainage of 9 square miles and contains 63,000,000 cubic feet of water. Between the two reservoirs there is a total fall of 475 feet, and from the lower reservoir to the tail-race at the power station, about 1¼ mile farther down stream, the fall is 222 feet. The natural flow of the stream is too small to be of practicable value for use all the year round, but by this system of storage enough water can be obtained during periods of rain or flood to give a steady working flow distributed over the whole year. At present the water from the upper reservoir follows its natural channel in passing to the lower reservoir, and no electric power is developed in this part of its course, but the complete scheme includes a pipe-line from this upper reservoir, which will be installed when the demand for power is too great to be supplied from the lower reservoir under the head of 222 feet.

The estimated rainfall over the area which drains into the two reservoirs is 45 inches per annum, and the storage capacity of the upper reservoir is equivalent to a depth of 11·5 inches over the area of 15 square miles which drains into it. Thus the reservoir may be filled if only 25 per cent. of the rainfall is retained, while measurements have shown the percentage of run-off from this area to be as much as 70.

The dam for the upper reservoir is 750 feet long, and has a maximum height of 54 feet above its foundation. The waste weir takes up 100 feet of the length of the dam, the remaining 650 feet acting as retaining wall. This weir is of stone masonry laid in cement; elsewhere the dam is of earth, with a masonry core-wall

rising from the foundation to 3 feet from the top of the earth embankment. The dam at the lower reservoir has a length of 585 feet, of which 111 feet are occupied by the spillway. The remaining 474 feet consists of an earth embankment, 31 feet high, containing for the greater part of its length a core-wall of stone masonry rising from the foundation to 3 feet from the top of the embankment. In its deepest part the embankment has a bottom width of 108 feet, the uniform width at top is 12 feet, and the slope on each face is 2 to 1. The core-wall is 30 inches wide at the top.

The hydraulic and electrical equipment at the power-station presents no novel features.

W. C. H.

Crushing Strength of Cement Mortar. O. CARLSEN.

(Teknisk Ugeblad, Christiania, 1905, pp. 333–6.)

As director of the Government Testing Laboratory at the Technical School, Christiania, the Author received frequent enquiries as to the influence of frost upon the hardening of cement. He therefore carried out during March—May 1905 a series of experiments upon hardening under various conditions; and tabulates the results at considerable length. Details are first given of the cement and sand employed, of the testing of each separately, of the proportions of their mixture, of the quantity of water used, and of the tensile and crushing strengths of samples of the cement-mortar. Altogether some 130 briquettes were prepared and tested, all well rammed and 7·1 centimetres (2·8 inches) long, mixed by weight 1 cement to 3½ sand with 7½ per cent. water. The testing was done in four groups, in which are tabulated in detail a couple of dozen tests in all, showing the crushing strength, the rates of hardening and drying, and the loss of weight in drying. In group 1 the briquettes were kept for the first day[1] in air at 15° to 18° C. = 59° to 64° F., and then twenty-seven days longer in water of this temperature. In group 2, after the first day in air of this temperature, they were laid on a shelf just under the roof in a store-room, where the temperature was the same at a height of 1 to 1½ metre (3 to 5 feet) above the floor; and they were not sprinkled with water. In group 3 they were first kept six hours in air of same temperature, and afterwards in air of 0° C. = 32° F.; and were not sprinkled with water. In group 4, after six hours in air at 15° to 18° C. = 59° to 64° F., the briquettes were kept in air of 0° C. = 32° F. from one up to fifteen days; then frozen for seven days in air of − 19° C. = − 2° F. (= 34 degrees F. of frost); then thawed for twelve hours in air of 10° C. = 50° F.; and finally kept in air of 0° C. =

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[1] Throughout this and the following abstract "day" means day-and-night of twenty-four hours.—A. B.

32° F. for twenty-one down to six days, so as to make up a total of twenty-eight days; they were not sprinkled with water. Arranging the four groups side by side in a comparative table, group 2 shows the poorest results. In the other three groups, apart from the seven days' freezing, the results agree so closely that the hardening seems to have gone on under exactly the same conditions; and the same strength has been attained by hardening in water of 15° to 18° C. = 59° to 64° F. as in air of 0° C. = 32° F. Longer freezing at − 19° C. = − 2° F. seems not to have had any other result than that the hardening manifested itself during the freezing period, even if the mortar was fresh when the frost got into it. At the same time it must be borne in mind that these briquettes were mixed with very little water, and both in mixing and in ramming were handled with special care; moreover the mixture was relatively rich, and the sand particularly good.

A. B.

Cement-Bricks and Sand-Testing. O. Carlsen.

(Teknisk Ugeblad, Christiania, 1905, pp. 434–5.)

Subsequently to the appearance of the article dealt with in the preceding abstract, the Author was further appealed to respecting the proportion of sand to be mixed with the cement for cement-bricks. In view of the overwhelming importance of the sand, he carried out for a number of years a series of tests comprising fifteen different kinds. After twenty-eight days' hardening of the briquettes, the best of those mixed in the proportion of 1 cement to 3 sand by weight gave about sixteen times the crushing strength of the poorest in the same proportion. As an example, he quotes in detail tests made in the spring of 1904, with three qualities of sand then in store, of which No. 1 seemed to be particularly good, No. 2 somewhat better than average, and No. 3 somewhat poorer than average but still not unfit for use. It would generally be expected that No. 1 must be the cleanest and No. 3 the most impure; but this was not so, for the loss by washing was 3 per cent. of No. 1, and 1·7 per cent. of No. 2, and 1 per cent. of No. 3. Besides neat cement, seven mixtures were tested:—namely, 1 cement by weight to 1–2–3–4–5–7–10 sand of each of the three kinds. The crushing strengths are tabulated, and show that practically there is no increase of strength from using more cement than sand; with No. 1 sand the strength of the mixture 1:1 is actually greater than that of neat cement, as has often been observed in other tests. Any desired crushing strength can be obtained with different mixtures of these three qualities of sand. Thus 1:10 of No. 1, and 1:4¾ of No. 2, and 1:3¾ of No. 3 will each alike give 100 kilograms per square centimetre (1,422 lbs. per square inch). The Author's experience, based upon several hundred tests, shows that the sand plays a far more important part than the cement. He decidedly advises discretion in

choosing not only the sand but also the proportion of mixture. Whether for cement-brickwork, or for concrete, or for anything else, the proportion cannot be determined without knowing the quality of the sand.

A. B.

Method and Cost of Constructing Cement Pipe in Place.

Halbert P. Gillette.

(Engineering Record, New York, 10 March, 1906, pp. 349–50.)

This is an invention of Mr. Ernest L. Ransome, and consists of building a cement pipe in its trench by means of a movable mould. The rate of progress, in the case of an 8-inch pipe at a building in Rochester, N.Y., was 300 lineal feet per 10-hour day, by six men and a foreman, three being in the trench moulding and three above mixing the mortar and delivering it.

The mould is of sheet steel, with an inner core 10 feet long, the front end of which is surrounded by a short steel shell that serves as the outer form for the cement pipe. The mortar is packed in between the inner core and this outer shell by one of the men with a rammer. Another man keeps moving the mould slowly forward by means of a lever grasped in the right hand, the lever being provided with a dog that works in a ratchet, and thus rotates a small drum on which a wire rope is wound. The rope is anchored ahead.

The cement mortar is mixed with a small amount of water, so that it possesses sufficient adhesion to hold together when unsupported by the core. To protect the pipe until it hardens, a little earth is packed round the sides and top. This and the back filling is done by the third man in the trench. The pipe does not break down of its own weight even when made 3 feet in diameter. To put in a branch, a hole is cut in the side of the green pipe before the core has been pulled ahead, and a branch of the proper pattern shoved up tightly against the pipe, and the collar of the branch is plastered with cement mortar.

The thickness of the pipe is not given, except as far as it can be inferred from the quantities, which, with the cost, are given as follows:—

	$
6 men at $1·70 per day	10·20
1 foreman	2·00
3 barrels cement at $1·25	3·75
3·30 cubic yards sand at $0·85	2·80
Water	0·15
Total 300 feet of 8-inch pipe	19·90 (*sic*) = £4 2*s*. 11*d*.

or 6·63 cents (*sic*) = 3·35*d*. per lineal foot.

A similar Table gives the cost of 12-inch pipe at 10·25 cents (5·125*d*.) per lineal foot, and these are compared with vitrified pipe of similar

diameter laid at 17·50 cents (8·75*d.*) and 35 cents (1*s.* 5½*d.*) respectively. The cost of trenching is not included in either case, but is said to be less in the case of the jointless pipe, as the width required is less. The risk of leakage of jointed pipes is also avoided. Illustrations help the description.

C. O. B.

Government Aid for Research Work on Concrete.

(Engineering Record, New York, 10 March, 1906, p. 327.)

This is an appeal for government assistance in laboratory work, to investigate, by extensive experiments, the properties of concrete and its various systems of reinforcement. It is shown to what an enormous extent structures exist to which this material is every day increasingly applied in the States, not only in engineering and architecture, but in farm buildings, etc., and the ignorance often displayed, resulting in unsafe building and danger to the public, is pointed out. There is at present a joint committee of numerous technical societies engaged in procuring data, and they have carried their work so far as to recognize that the further development of the necessary tests which the country's interest demands depends on a liberal Federal grant—$100,000 (£20,833) per annum for 2 years is asked for.

The American Society of Civil Engineers has had a committee working for some years to determine the best methods of testing cement, but its report is not yet issued. The Government now has, at St. Louis, a well-equipped laboratory, where some investigations on concrete have been carried on, and the importance of the work has led the President to appoint recently a committee of engineers to supervise this research, so it is hoped that Congress will furnish the means.

The article concludes by estimating that, with the help of Congress, the desired data may be had in accurate form in 2 years. Without such assistance, the information will be many years delayed in its appearance, and the dangers and extravagance which characterize many features of concrete construction to-day will continue indefinitely.

C. O. B.

Concrete Aggregates.

(Engineering Record, New York, 27 January, 1906, pp. 108–10.)

Using the term aggregates as denoting the stone and sand, the Author gives some valuable information, mostly based on carefully carried out tests, as regards, (1) The effect of different aggregates when mixed with cement in the same proportions; (2) the selection

of stone; (3) the value of different stone; (4) the selection of sand; (5) the testing of sand; and (6) the strength of mortar.

Under the third heading, the order of merit of the different kinds of stone is given as (1) trap; (2) granite; (3) gravel; (4) marble; (5) limestone; (6) slag; (7) sandstone; (8) slate; (9) shale; and (10) cinders, the last three differing very slightly.

Under the fourth heading, much is said about the percentage of voids in the different qualities of sand, and also about what the Author thinks is undesirable, viz., the sharpness generally specified; he finding, on the contrary, that the sand with rounded grains gives the best results. As to the fifth heading, the modes of carrying out the tests, measuring, weighing, etc., are given in considerable detail, and in the sixth paragraph, closing a valuable contribution to engineering knowledge, a formula is given for the compressive strength of mortar, the data given being the volume of the mortar, the weight and specific gravity of the cement and of the sand, and a constant varying with the different kinds and age of the cement.

C. O. B.

Reinforced Concrete for Columns.

(Engineering Record, New York, 10 February, 1906, pp. 165–6.)

This is a valuable Paper on the behaviour and durability of armoured-concrete columns by Mr. James E. Howard, of Watertown Arsenal, where much recent experimenting has been done. The Author first considers the effect of hooping in the reinforcement, as providing against the lateral yielding of the material, and deals with the unavoidable necessity of assembling the hoops when the concrete is still plastic, with the effects of increase in the volume of the cement which accompanies hydration, and with shrinkage, and shows that hooping is of service in tending to keep in place the material after local disintegration is well advanced. The effect of this system with lean and rich concretes is discussed at considerable length. The Author then passes on to these and other considerations in the case of reinforcement by longitudinal steel bars. The occasional use of horizontal iron plates for the distribution of the stresses is advised in the case of long columns, and the varying ratio of stress per square inch on the steel and on the concrete, according to the induration consequent on the age of the latter, is referred to. The durability of the concrete under the two methods is then treated, and owing to the hoop system allowing stresses on the concrete to become active without its support earlier than in the longitudinal arrangement, the Author decides on the superiority of the latter in this respect.

C. O. B.

Berlin Testing-Establishment for Hydraulic and Naval Engineering. EGER, DIX and SEIFERT.

(Zeitschrift für Bauwesen, Berlin, vol. lvi., Heft i-iii. pp. 123–72 *et seq.*)

A sketch-plan indicates the position of the various buildings, which are situated on an island adjoining the Landwehr Canal, between Berlin and Charlottenburg, in the vicinity of the Technical High School. It is pointed out that the presence of a large volume of water at the intake of the canal, with a constant head of about 5 feet, and amounting to many hundreds of gallons per second, is very valuable for experimental purposes. Among the buildings mention is made of a covered basin of about 492 feet to 525 feet available length, 23 feet to 26·2 feet in width, and 10 feet 6 inches in extreme depth for experiments with vessels, flotation, resistances, &c.; a propelling frame for towing models, testing propellers and measuring apparatus in the basin; a workshop for the construction of models of ships, screws and testing work in general; a channel 59 feet to 65 feet in length, 6·5 feet wide and 1 foot in depth for arranging model water-courses, and for carrying out experiments in the flow of water; a conduit for bringing the water from the Landwehr Canal into the large basin for experiments in the flow of water on a more important scale. There is an electrical power-plant for working the towing apparatus, running the workshops, driving a special pump for the filling and emptying of the large basin, and for conveying water to the small channel; three or four working-rooms, and the requisite buildings for the heating, lighting, water-supply and lavatories. Sections are given to indicate the proportions of the principal basin, as compared with those at Dumbarton, Haslar, Spezia, Washington, Bremerhafen, etc. The cost of the various works and buildings is estimated at £19,100. Considerable economies in the expenditure have been rendered possible by the use of the railway arches under the line crossing the site. Descriptions with photographic illustrations are given of the various arrangements and details of the equipment.

G. R. R.

Swedish Trials of Coaling Peat in Heaps.

(Teknisk Tidskrift, Stockholm, 1905, General section, pp. 365–8.)

Sweden's northernmost county is Norbotten (latitude 65°–69°); and its northern division, wherein are situated the Gellivare iron-ore mines (latitude 67° 18′), is the region richest in ores, but is either poorly timbered or treeless. The great gain that would result from exporting pig-iron, instead of iron-ore, led the government to carry out trials of coaling peat at Koskivara in the summer of 1904, which were reported by the Royal Board of Agriculture in 1905. Coaling peat in heaps is not so advantageous as in ovens, because

the latter can be worked continuously and quicker and under better control, and yield a higher quality of charcoal, besides valuable by-products. But ovens, though cheaper and better in working, especially on a large scale, involve much greater outlay; whereas all that is wanted for heaps is to prepare a dry floor for building them on, and to cover them air-tight when built up, and perhaps also to screen them from wind. Cost of carriage is reduced to a minimum; and heaps can be worked close to peat bogs. The object of the Koskivara trials was to ascertain the practicability of coaling peat in Norbotten, and to get samples for analysis from Majenjänkä bog, and to see whether the charcoal made therefrom was as strong for the blast-furnace as wood charcoal. Three heaps were tried: the first contained only 1 ton of peat, after which the two others were built of 3·1 and 3·6 tons; the whole quantity was 7·7 tons; and the trials cost altogether 62·10 kronor (£3 9*s.*), exclusive of the cost of the peat.

The official report furnishes ample details under the following headings:—Nature of peat employed; position of heaps, and preparation of floors; building of heaps, kindling, and process of coaling; quenching the charcoal; yield of charcoal; cost of coaling; analyses and testing of charcoal. The Majenjänkä peat, cut in 1903 and well air-dried, was in billets 80 × 85 × 200 millimetres ($3\frac{1}{8}$ × $3\frac{1}{3}$ × 8 inches); it weighed about 34–35 kilograms per hectolitre (say 27–28 lbs. per bushel), and a perfectly dried sample contained 5–6 per cent. ash. The three heaps were from 2 to 4 metres diameter at bottom ($6\frac{1}{2}$ to 13 feet), and 0·5 to 1·3 metre at top (20 to 50 inches), and 1·7 to 2 metres high ($5\frac{1}{2}$ to $6\frac{1}{2}$ feet). For coaling on a larger scale they should be much bigger, say up to 30 tons per heap. The smallest heap was fired at the top, to coal downwards, which seems the better plan for coaling peat; the two larger at the bottom, coaling upwards, as usual in coaling wood. The smallest stood two hours for the fire to spread, and then two more for "sweating," after which the coaling proper began; the two larger required yet two hours longer before the coaling could begin. The coaling took four days and nights in the smallest heap, and nine to ten in the two larger. Quenching is the most difficult stage of coaling peat in heaps. Peat charcoal, keeping hot much longer than wood charcoal, catches fire much more readily; and cannot advantageously be quenched with water, because the steam then splits it. If the heap be left to smother itself, there is risk of the charcoal being wasted by air leaking in. At Majenkändä, where no clay could be found for luting the heaps, they were coated with a layer of peat mud, wet and plastered close, which proved an effective covering. Owing to waste both in coaling and in quenching, the yield from the two larger heaps was only 22 per cent., namely 13 per cent. charcoal and 9 per cent. half-coaled peat. The smallest gave a normal yield of 30 per cent., namely 24 per cent. charcoal and 6 per cent. half-coaled. Data as to cost cannot be got from these small trials; but during 1890–1900 the Swedish Peat Charcoal Committee reckoned 5 kronor as the working expenses per ton of charcoal when made on a large

scale; adding 18 kronor for 3 tons of peat coaled, the total cost would be about 23 kronor (25*s.* 7*d.*) per ton of charcoal, exclusive of carriage to store and railway and consumer. Wood charcoal averaging 14 kilograms per hectolitre (11·2 lbs. per bushel, or 10 bushels per cwt.) was fetching 32 kronor per ton (35*s.* 7*d.*) at the ironworks in 1903. The Majenjänkä peat charcoal was nearly twice as heavy, averaging 27·5 kilograms per hectolitre. A fairly good bog furnishes peat for more than one hundredfold as much charcoal as the wood charcoal that could be made from the same area of forest. Average samples of the better portions of the peat charcoal from the Majenjänkä heaps were analysed in four laboratories. The heating-power, estimated after deduction of water and ash, ranged from 6,740 to 7,169 thermal[1] units; and the crushing strength from 36·2 to 55·8 kilograms per square centimetre (515 to 795 lbs. per square inch). While so strong, the peat charcoal contains too much ash, sulphur, and phosphorus, to replace wood charcoal in the larger metallurgical processes; for smiths' forges, for locksmiths' and coppersmiths' work, and for other work needing scarcely a red heat, it can be used well. Peat coaling can therefore hold its own as a home industry. Sweden has plenty of peat bogs containing scarcely 2 per cent. ash, many of which probably contain so little sulphur and phosphorus that the charcoal they yield might be used in blast-furnaces when mixed with wood charcoal.

A. B.

Fixation of Nitrogen from the Atmosphere. E. Lemaire.

(Le Génie Civil, Paris, vol. xlviii. pp. 308-30.)

The rapid depletion of the natural beds of nitrates in Chili has made apparent the necessity of renewing the supply by artificial means. The atmosphere contains a very large proportion of nitrogen, and may be looked on as an unlimited source of supply; but the difficulty of combining the element in some form other than the gaseous has until recently been regarded as commercially impossible. The Author describes four methods which are now in daily use for fixing the atmospheric nitrogen; all of these require either directly or indirectly the disruptive electric discharge. In the first process mentioned advantage is taken of the fact that calcium carbide, on being heated to a temperature of about 800° C. in the presence of nitrogen, is decomposed, giving rise to a product, named by the discoverers Nitro-Lime, the formula of which is $CaCN_2$. By the action of water, preferably under pressure, the nitro-lime splits up into ammonia and calcium carbonate. Thus the nitrogen is fixed in the form of ammonia. The process is carried on in Italy, where, for the continuous expenditure of 3,000 HP., an output of 4,000 tons of nitro-lime can be secured per annum. The remaining three

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[1] Presumably on the Centigrade scale, though not so stated.—A. B.

methods all depend on the fact that when a disruptive discharge is passed through air, or, better still, through a mixture of one volume of nitrogen to two of oxygen, the two elements combine, and the nitrogen can be thus obtained in the form of nitric and nitrous acids. The three methods, which are in use principally in Norway, differ from each other in detail only, and the weight of nitrogen fixed varies from 126 kilograms to 275 kilograms (277 lbs. to 605 lbs.) per HP. per annum. The article ends with a theoretical examination of the possibilities of the methods last referred to, in which it is pointed out that the combination of nitrogen and oxygen is probably brought about by heat alone; but that immediately after combination the heat must be dissipated at once, otherwise decomposition will again take place. The electric spark gives a ready means of intensely heating the medium through which it passes, and, owing to the small volume acted on, the normal temperature is resumed with great suddenness. The efficiency depends on the use of high temperature and quick cooling.

I. C. B.

Deflection Potentiometer. H. B. Brooks.

(Electrical World, New York, 1906, vol. xlvii. pp. 564–5.)

The Author describes a deflection potentiometer devised by him, which has been constructed in the United States Bureau of Standards and used for voltage measurements in photometry. With this instrument the null method is partly replaced by a deflection method. An approximate balance is first obtained in the ordinary way, and thereafter the deflection of the galvanometer gives a measure of the difference between the potential indicated by the position of the slider on the bridge and that to be measured. Thus, for example, if an electromotive force to be measured is between 110 and 111 volts, the ordinary null method may be used to measure 110 volts, and the remaining fraction of a volt is then read from the galvanometer. The Author discusses the conditions which must be fulfilled by such a deflection instrument to give correct readings, whatever be the value of the electromotive force to be measured, and shows how his instrument is made to satisfy these conditions.

W. C. H.

Respiratory Apparatus for Use in Non-Respirable Atmospheres. Guglielminetti.

(Comptes Rendus de l'Académie des Sciences, Paris, 1906, vol. cxlii. pp. 60–3.)

The Author describes an apparatus, devised by himself and recently approved by the French Commission on Fire-Damp, which is intended to make the wearer independent of the ordinary

atmosphere for his supply of air to breathe, and thus to enable him to work without inconvenience for a period of two hours when surrounded by an atmosphere which cannot be breathed. The apparatus consists of three parts: (1) a cylinder containing 120 litres (about 4¼ cubic feet) of oxygen under a pressure of 120 atmospheres; (2) a regenerator and refrigerator; and (3) a respiratory mask. When the apparatus is in use, the exhaled air passes into the regenerator, where it is made to pass over trays containing caustic potash in small pieces and is deprived of the carbonic acid and of the ptomaines. In this process the air becomes heated and it is therefore passed next between the walls of two coaxial cylinders of sheet-iron, and is thus cooled. The tube which leads the air, thus purified but weak in oxygen, from this refrigerator is joined by one from the oxygen cylinder, which gives, by regulation of the nozzle, a steady flow of 2 litres of oxygen per minute, and the two streams of gas on mingling together give an atmosphere which can be breathed freely and without effort. The tube carrying this stream of pure breathing air passes into the mask near the mouth of the person wearing it. The mask is also fitted with two flat air-tight sacks or bags, one forming a reservoir for exhaled air on its way to the regenerator, the other communicating with the tube leading in the restored air and forming thus a reservoir for air to be breathed. Two light mica valves, one placed between each sack and the mask, and one opening inwards while the other opens outwards, keep the incoming current of fresh air from direct communication with the outgoing current of exhaled air. The mask is fitted over the head of the workman and is made air-tight by means of air-cushions. There is a mica window in front. The two flat sacks hang down over the chest of the workman, while the remainder of the apparatus is supported on the back by braces and a belt. The weight of the total apparatus is about 29 lbs. The size of the tubes and energy of the compressed oxygen are arranged to allow of the complete circuit through the apparatus eight or ten times per minute of the volume of air in the lungs (about 6 litres or ⅕ cubic foot) when the mask is first fitted. By this means a supply of air for 2 hours can be maintained.

W. C. H.

INDEX

TO THE

MINUTES OF PROCEEDINGS,

1905–1906.—PART III.

N.B.—Titles in italics refer to Original Papers, those selected for printing only being further distinguished by the suffix "(S.)." Abstracted Papers are not so indicated.

468

LONDON:
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p.468a

PLATE 1.

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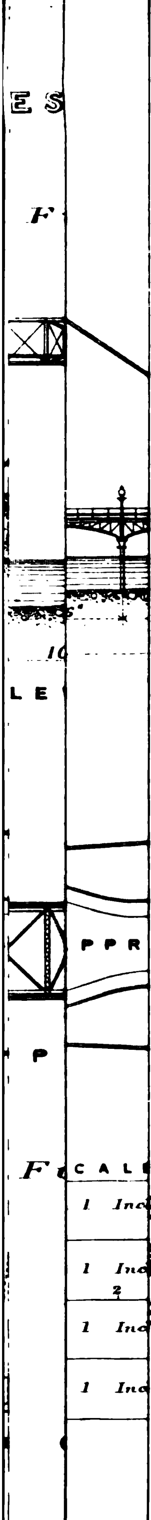
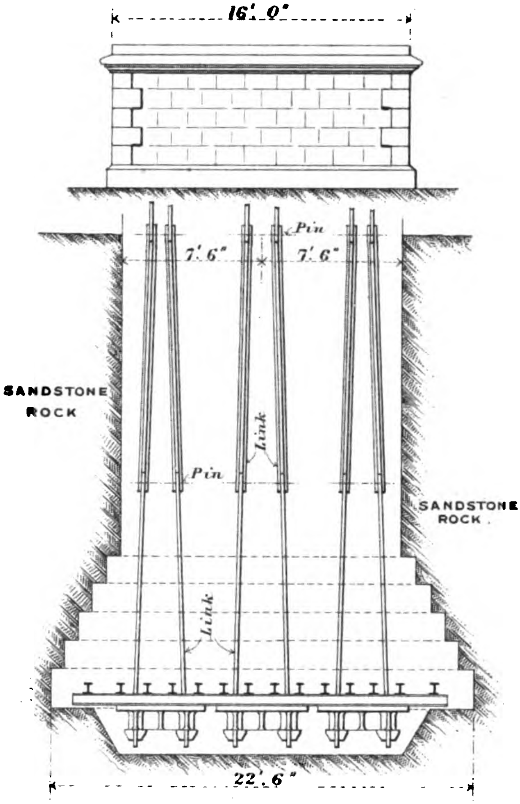
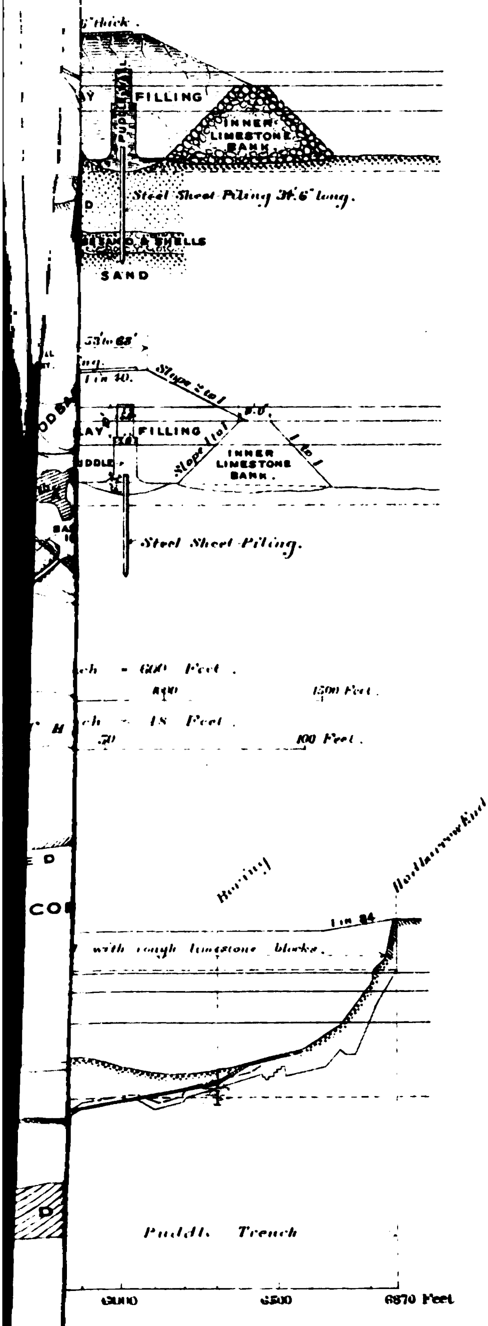




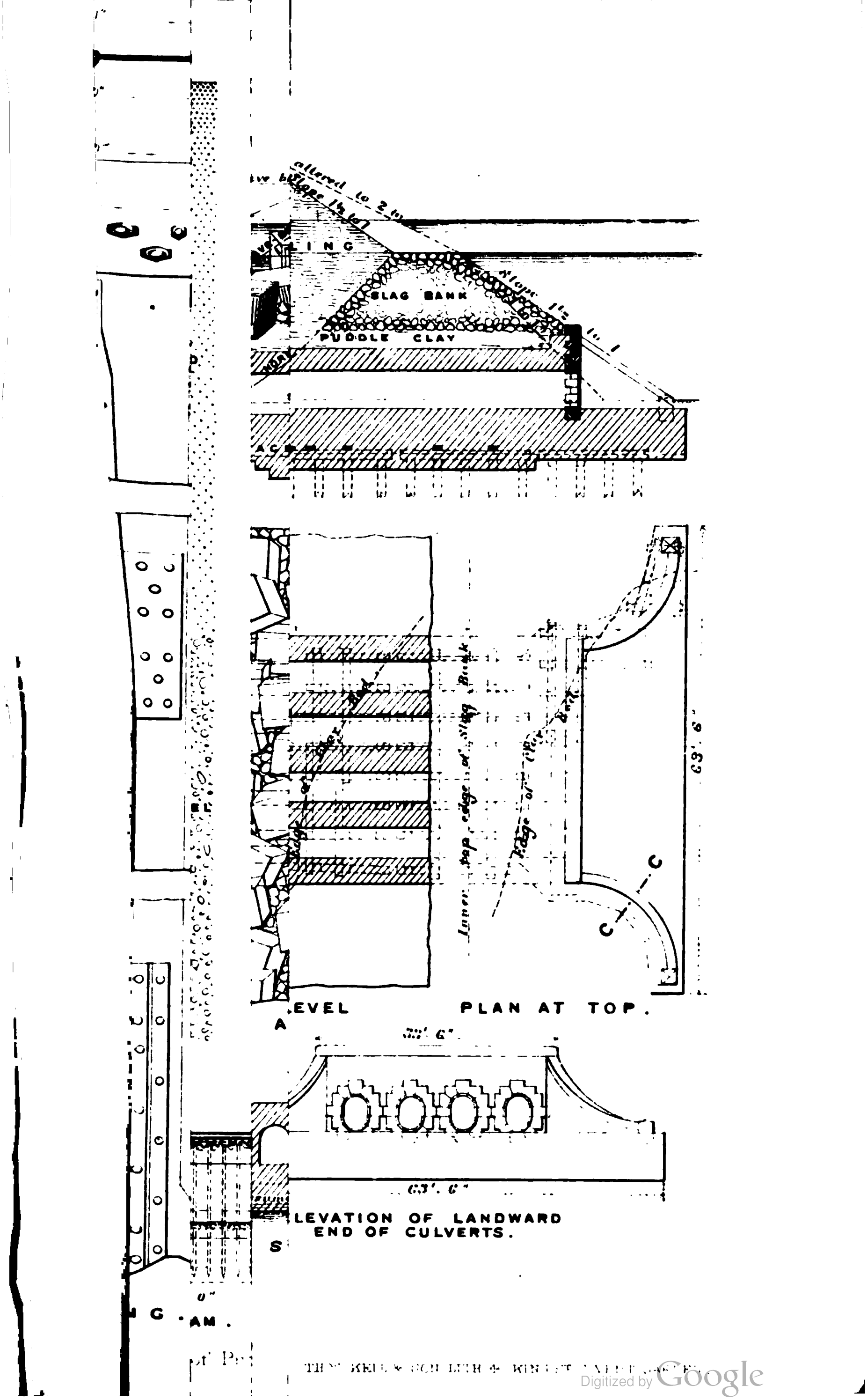
END SECTION.

THOS KELL & SON, LITH. 40 KING ST COVENT GARDEN.

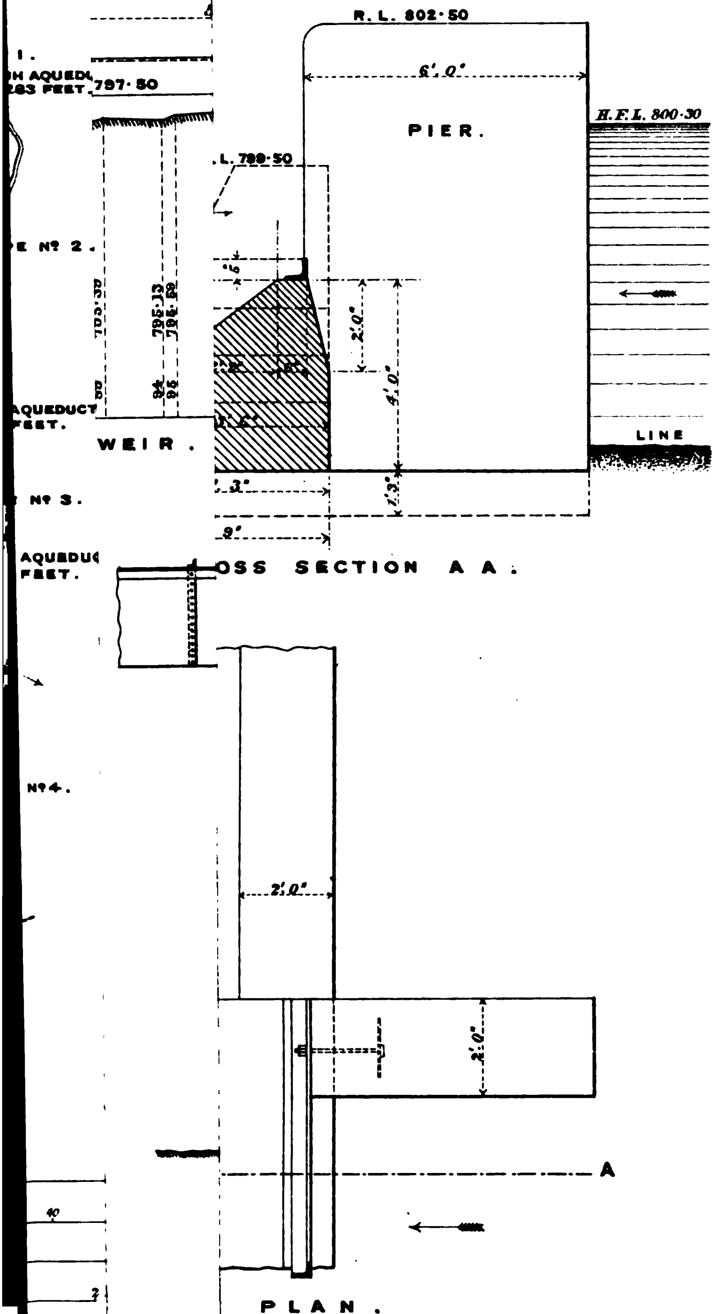
PLATE 4.



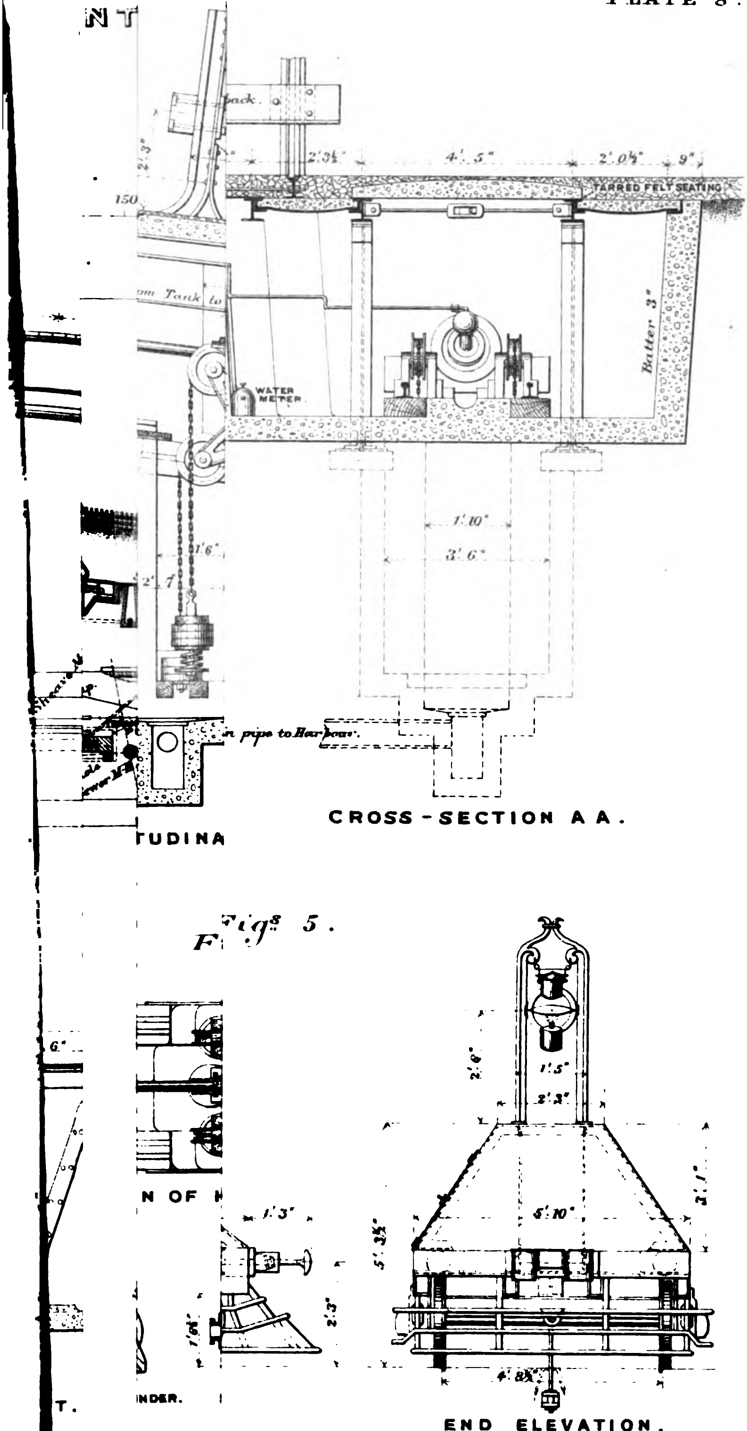
KELL ... LITH. 40 KING ST COVENT GARDEN.


altered to 2 to 1
Slope 1½ to 1
SLAG BANK
PUDDLE CLAY
LEVEL
PLAN AT TOP.
Inner top edge of Slag Bank
Edge of Clay Bank
63'. 6"
C
C
ELEVATION OF LANDWARD
END OF CULVERTS.
63'. 6"

Figs 4.



THOs KELL & SON, LITH. 40 KING ST COVENT GARDEN.



CROSS-SECTION A A.

Figs 5.

END ELEVATION.

FER-TROLLEY.

THOS KELL & SON LITH 40 KING ST COVENT GARDEN